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US Army Corps
of Engineers

# AGITATION DREDGING: LESSONS AND GUIDELINES FROM PAST PROJECTS

ORIGINAL

by

Thomas W. Richardson

Hydraulics Laboratory

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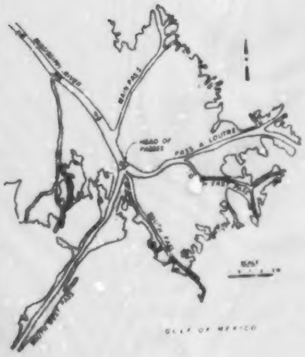
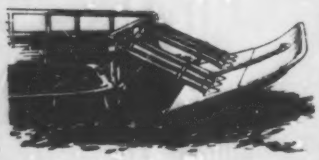
DEPARTMENT OF THE ARMY
Waterways Experiment Station, Corps of Engineers
PO Box 631
Vicksburg, Mississippi 39180-0631





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This report is based on a comprehensive literature survey of agitation dredging projects and the field data obtained from a small-scale monitoring study. Twenty-two agitation dredging projects are described from the standpoints of technical performance, environmental effects, and economic results. The projects encompassed seven different categories of agitation dredging equipment and several combinations of physical environment and sediment characteristics. The information gathered from these projects is used to develop

(Continued)

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20. ABSTRACT (Continued).

a number of general and specific conclusions about agitation dredging and the types of applications for which it is best suited. Several categories of agitation dredging equipment are selected as having the most technical potential. The author recommends that existing agitation dredging technology be improved and standardized and that a comprehensive approach be developed for planning and conducting agitation dredging.

# PREFACE

Investigations described in this report were accomplished under the auspices of the New Dredging Concepts project using funds authorized by the Office, Chief of Engineers (OCE), US Army, through the Improvement of Operation and Maintenance Techniques (IOMT) research program. Messrs. James L. Gottesman (OCE) and Charles Hummer (Water Resource Support Center) are Technical Monitors of the IOMT research program.

This study was conducted in the Hydraulics Laboratory of the US Army Engineer Waterways Experiment Station (WES) under the general supervision of Messers. H. B. Simmons, Chief of the Hydraulics Laboratory; F. A. Herrmann, Jr., Assistant Chief of the Hydraulics Laboratory; R. A. Sager, Chief of the Estuaries Division; and E. C. McNair, Jr., Chief of the Sedimentation Branch.

Mr. J. C. Roberge, former Research Hydraulic Engineer, Estuaries Division, supervised WES activities in the Harbour Town Marina experiment and gathered much of the information used in this report. Mr. T. W. Richardson, Research Hydraulic Engineer, Estuaries Division (presently Chief of the Coastal Structures and Evaluation Branch, Coastal Engineering Research Center), analyzed the information and wrote this report.

Commanders and Directors of WES during this investigation and the preparation and publication of this report were COL Nelson P. Conover, CE, and COL Tilford C. Creel, CE. Technical Director was Mr. F. R. Brown.

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# CONVERSION FACTORS, US CUSTOMARY TO METRIC (SI) UNITS OF MEASUREMENT

US customary units of measurement used in this report can be converted to metric (SI) units as follows:

| Multiply | By | To Obtain |
|---|---|---|
| cubic feet per minute | 0.02831685 | cubic metres per minute |
| cubic feet per second | 28.32 | litres per second |
| cubic yards | 0.7645549 | cubic metres |
| feet | 0.3048 | metres |
| feet per second | 0.3048 | metres per second |
| gallons per minute | 0.06308 | litres per second |
| horespower | 0.7457 | kilowatts |
| inches | 25.4 | millimetres |
| knots | 1.825 | kilometres per hour |
| miles (US statute) | 1.609 | kilometres |
| million gallons per day | 43.8056 | litres per second |
| pounds (force) per square foot | 47.8803 | pascals |
| pounds (force) per square inch | 6894.757 | pascals |
| pounds (mass) | 0.4535924 | kilograms |
| pounds (mass) per cubic foot | 16.01846 | kilograms per cubic metre |
| square feet | 0.09290 | square metres |
| square yards | 0.8361 | square metres |
| tons (2,000 lb, mass) | 907.1847 | kilograms |

# AGITATION DREDGING: LESSONS AND GUIDELINES FROM PAST PROJECTS

## PART I: INTRODUCTION

### Definition

1. "Agitation dredging" is a term that is partly self-descriptive. Many persons will concur on a general meaning but will have trouble arriving at an exact definition. Dredging literature is of little help in this regard as even the textbooks (Huston 1970; Cooper 1975; Herbich 1975) discuss agitation dredging historically but offer no definitions. The Office, Chief of Engineers (1953) defines hopper dredge agitation as "a process which intentionally discharges overboard large quantities of dredged material with the objective in view that a major portion will be transported and permanently deposited outside the channel limits by tidal, river, or littoral currents." This single purpose definition alludes to the two distinct phases of agitation dredging which most writers agree must be present: (a) suspension or resuspension of bottom material by some type of equipment and (b) transport of the suspended material by currents. These two phases in turn suggest the following general definition of agitation dredging:

> The removal of bottom material from a selected area by using equipment to raise it temporarily in the water column and currents to carry it away.

2. This definition touches on a number of points important to subsequent discussions in this report:

- a. Agitation of the bottom material is accomplished by some type of equipment. This excludes periodic scouring of the bottom by tidal currents, river or lock discharges, etc., from the subject of agitation dredging.
- b. The main purpose of the dredging equipment is to raise bottom material in the water column. Currents are used to move the material away from the dredging site. Natural currents are usually involved in this transportation, although they may be augmented by ones generated by the agitation equipment. Although shallow-draft sidecasting dredges and dustpan dredges do not strictly adhere to this aspect of the definition (they move material horizontally by pipe before releasing it into the water column), they are discussed briefly in Appendix B of this report.

c. Since currents are a necessary part of the agitation dredging process, a sound understanding of local hydrodynamics is needed for a successful operation.

d. "Removal of bottom material from a selected area" is the essence of the definition. If the material is suspended but redeposits shortly in the same area, only agitation (not agitation dredging) has been accomplished. Therefore some degree of success is inherently required for an operation to be called agitation dredging. This requirement places agitation dredging on an equal basis with other types of dredging, which can be relied on to remove some material regardless of cost or operational difficulties. When agitation dredging is attempted, however, even limited success is achieved only when the operation is coordinated well with local physical processes.

## Objectives

3. The decision on whether to employ agitation dredging in a given situation should be based primarily on the following factors:

a. Technical feasibility--is equipment available which can generate the required level of agitation, and will the agitated material be carried away by currents?

b. Economic feasibility--is agitation dredging the most effective method for achieving the desired results? How will agitation dredging affect the costs of other types of dredging in the area?

c. Environmental feasibility--will agitation dredging cause unacceptable water quality changes or biological effects?

4. Factors b and c are highly job-specific. Factor a also is to a large degree, but the merits of a technique or piece of equipment can be discussed in some detail without a site in mind. As a first step toward a comprehensive method of evaluating these factors for a particular project, past agitation dredging projects can be analyzed for specific lessons or general guidelines which may be present.. Such an analysis can also identify areas where investigation or development could be beneficial. Therefore the objectives of this report are to: (a) develop lessons and guidelines for agitation dredging from a study of past projects and (b) determine whether further application and development of agitation dredging is justified.

## Approach

5. A number of agitation dredging projects covering a wide range of equipment and conditions are presented in Appendix A. Table 1 summarizes

## Table 1

## Summary of Example Agitation Dredging Projects in Appendix A

| Location | Appendix Paragraph Numbers | Equipment Type | Environment Type | Sediment Type | Environmental Monitoring? | Economic Data? |
|---|---|---|---|---|---|---|
| Eastham Channel | A2 | Hopper dredge | Estuary | Sand | No | No |
| Harbour Town Marina | A3-A11 | Air bubbler | Sound | Silt | Turbidity | No |
| Delaware Estuary | A12-A21 | Hopper dredge | Estuary | Silt, clay, and fine sand | No | Limited |
| Chinook Channel | A22-A30 | Propwash | Estuary | Sand | Yes | Yes |
| Missouri River | A31-A35 | Propwash | River | Sand | No | No |
| Pacific Northwest (nine projects) | A36-A42 | Propwash | Estuary, river, coastal harbor | Sand, sand and gravel, silt and fine sand | No | Yes |
| Tillamook Bay | A43-A53 | Propwash | Estuary | Sand | Yes | No |
| Savannah Harbor | A54-A73 | Drag beam | Estuary | Silt | Yes | Yes |
| Grays Harbor | A74-A85 | Vertical mixers | Estuary | Silt, clay, and sand | Limited | No |
| Mare Island Naval Shipyard | A86-A117 | Water jets, propwash, vertical mixers, air bubblers | Estuary, laborabory | Silt and clay | No | Yes (water jets only) |
| People's Republic of China | A123-A126 | Rakes | Estuary, laboratory | Fine grained | No | Limited |
| Louisiana Gulf Coast (two projects) | A127-143 | Hopper dredge | Estuary, approach channel | Silt, clay, and fine sand | Yes (one project) | Yes |
| Mare Island Strait | A144-148 | Hopper dredge | Estuary | Silt and clay | No | No |

Note: In this table and in text, paragraphs in Appendix A are referenced as "paragraphs A3-A11," etc., though "A" is not actually used before paragraph numbers in the appendix.

these projects and their characteristics. In PART II of this report, the factors of technical, economic, and environmental feasibility will be discussed in terms of these projects. As might be expected, a relatively small amount of information is available in the literature on economic feasibility, although additional comparisons with other types of dredging can be inferred. Somewhat more has been written about the environmental effects of agitation dredging, and the largest body of information by far deals with technical aspects of the subject.

6. The three factors will be discussed in their logical order of consideration: technical, environmental, and economic. If a technically feasible agitation dredging method cannot be found for a project, the other two factors are irrelevant. Since economic feasibility is the most relative of the three factors, it need only be applied to those methods which appear both technically and environmentally sound.

7. PART II was written assuming that the reader is somewhat familiar with the referenced agitation dredging projects. THEREFORE THE READER SHOULD REVIEW THE EXAMPLE PROJECTS PRESENTED IN APPENDIX A BEFORE READING PART II OF THIS REPORT. Appendix B provides additional background to PART II by describing the dustpan and sidecaster dredges.

## PART II: FEASIBILITY

8. "Feasibility" has different shades of meaning when applied to technical, environmental, and economic factors. Technical feasibility is the most straightforward: can a system perform the required function(s) and if so, how well? If knowledge of the subject were sufficient, environmental feasibility would be a question largely of meeting certain quantifiable standards and regulations. At present, however, the perception of possible environmental effects is often more important in determining feasibility. Economic feasibility is usually defined relative to other methods available, although there may be instances where absolute criteria are set. Discussions in PART II will be based on these somewhat different aspects of feasibility.

### Technical

9. Technical feasibility for an agitation dredging method must consider not only the equipment employed but the conditions under which it is used. These conditions include currents, water depths, wave action, bottom sediment characteristics, and other aspects of the natural regime where agitation dredging is attempted. Failure of agitation dredging at a particular site may be due to a fault inherent in the equipment itself or to the misapplication of that equipment.

10. The types of equipment or methods covered in Appendix A are summarized in Table 1. Although Appendix A is by no means a complete presentation of past agitation dredging projects, it does contain examples of most types of agitation dredging equipment or methods. Technical feasibility will be discussed in terms of these different types.

#### Hopper dredges

11. Hopper dredge agitation is of two types: (a) intentional agitation produced by hopper overflow and (b) auxiliary agitation caused by dragheads and propeller wash. Since the latter is present in all hopper dredge operatious and since it is difficult to quantify separately from hopper overflow, both types are invariably measured together in any study of hopper dredge agitation effects.

12. Four projects involving hopper dredge agitation are discussed in Appendix A: Eastham Channel, Delaware Estuary, Louisiana Gulf Coast, and

Mare Island Strait. The Louisiana Gulf Coast section is divided between Southwest Pass and the Calcasieu River, and the Mare Island Strait section contains a short reference to Pinole Shoal Channel dredging.

13. Southwest Pass and the Calcasieu River (paragraphs A127-A143) are by far the largest ongoing agitation dredging projects in the United States. Neither channel could be maintained using the same amount of equipment if agitation dredging were discontinued; in fact, during periods of high shoaling rates, agitation dredging is essential even when additional dredges are assigned to the projects. The technical feasibility of these two projects is due to a number of factors: (a) in the loosely consolidated, fine-grained sediments found at these projects, the hopper dredge can easily raise bottom material to the surface, (b) strong surface river currents and transverse littoral currents at Southwest Pass work together to help remove agitated material from the navigation channel, (c) at the Calcasieu River, agitated material needs to travel only a relatively short distance to settle out of the channel, (d) the hopper dredge *Langfitt* can overflow material near the water's surface instead of from the tops of the hoppers, and (e) both projects have been in existence long enough to allow adjustments and improvements in operating methods.

14. The Mare Island Strait project (paragraphs A144-A148), although ultimately successful as a combination of agitation dredging and hauling, points out the need for coordination with local processes and adjustments in operating methods even when conditions are favorable for agitation dredging. Relatively minor revisions in the dredge operating schedule to take advantage of ebb current variations and minimize the undesirable effects of flood currents changed the project from unfeasible to feasible. A short distance away in Pinole Shoal Channel, however, different sediment characteristics made agitation dredging unfeasible under any circumstances.

15. Southwest Pass, the Calcasieu River, and Mare Island Strait illustrate a fact about technically feasible hopper dredge agitation projects: they allow navigation channels to be maintained with relatively small hopper dredges. The *Langfitt* is a medium capacity hopper dredge by today's standards, and the *San Pablo* would be almost a miniature. Neither one could have succeeded in its assignment in a conventional dredge-haul-dump operating mode. In agitation dredging, however, hopper capacity is of secondary importance compared with pumping rate, mobility, and overflow provisions. Even smaller hopper dredges may have sufficient pumping ability to handle an agitation

dredging project; Figures 1 and 2 support this point. Figure 1 shows total installed pump power versus hopper capacity for 38 hopper dredges ranging in size from 500- to 13,000-cu-yd* capacity. Figure 2 shows the same data expressed as the ratio of pump power to hopper capacity versus hopper capacity. Figure 2 divides neatly into several zones as shown by the dotted lines. All of the listed hopper dredges above 4,500-cu-yd capacity have power:capacity ratios less than 0.53, while less than 40 percent of those smaller than 4,500 cu yd fall below that line. Ninety-three percent of those above 0.53 are smaller than 3,200-cu-yd capacity, while 76 percent of those below 0.53 are greater than 3,200 cu yd. While the exactness of these figures is doubtful, the trend they indicate is clear: small-to-medium size hopper dredges are more likely to have greater installed pump power for their size than larger hopper dredges. Both the *Langfitt* and the *San Pablo* are seen to have high power:capacity ratios. In Figure 1, the *Langfitt* is shown to have a

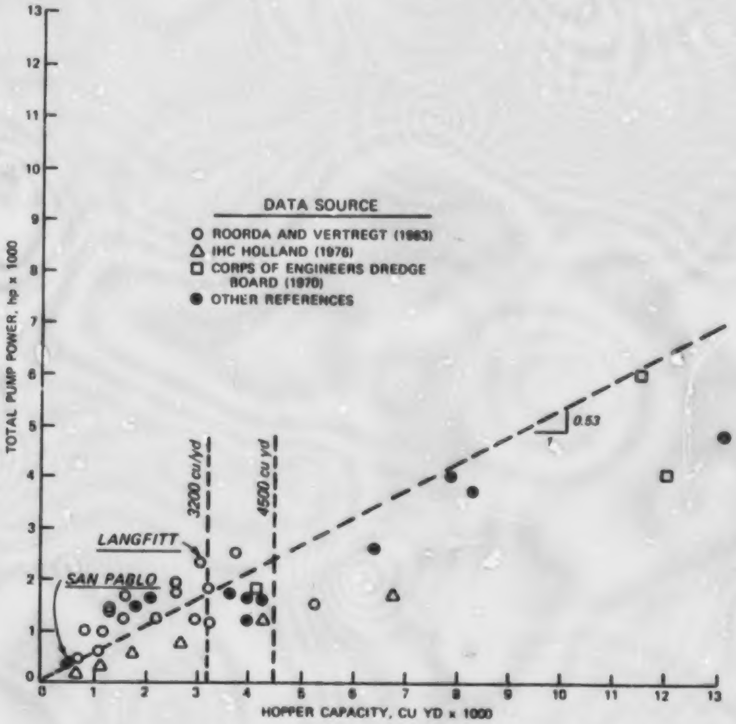

Figure 1. Pump power versus hopper capacity, representative hopper dredges

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* A table of factors for converting US customary units of measurements to metric (SI) units is presented on page 3.

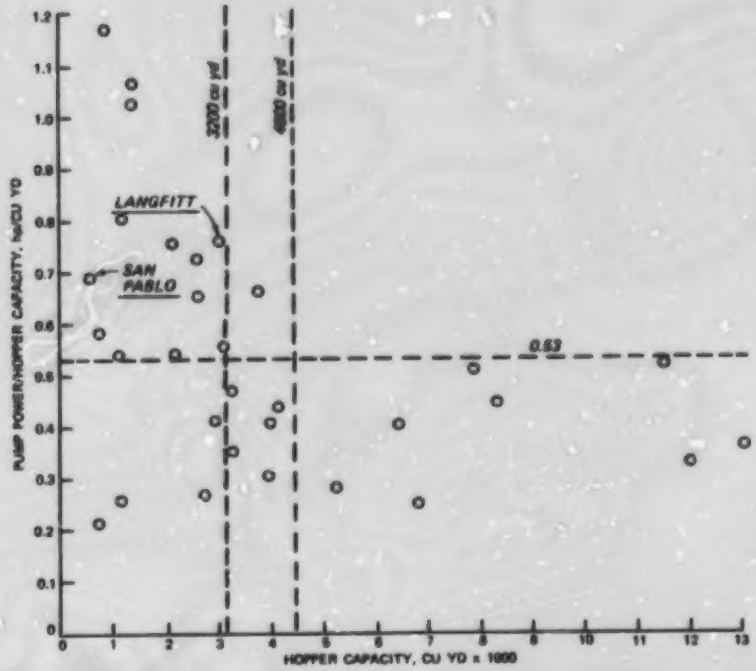

Figure 2. Pump power/hopper capacity ratio versus hopper capacity, representative hopper dredges

total pump power greater than 10 larger listed dredges up to twice its capacity. Although other factors such as pump efficiency and suction side head losses help determine a dredge's actual pumping capacity, the total power connected to its pumps can serve as a gross measure of relative capabilities.

16. The Delaware Estuary agitation dredging experience (paragraphs A12-A21) can be viewed as a classic example of the need for complete understanding of project hydrodynamics in agitation dredging. At first glance, the project would appear to have all the attributes for success: fine sediments, a well-mixed salinity regime, reasonable ebb current velocities, and equipment more than adequate for the task. The tremendous dredging effort expended in the period 1905-1954 should have been able to remove existing sediments and keep ahead of new influxes, if the agitated sediment could have been gradually "flushed" from the estuary. However, the estuary flow regime made such a mechanism impossible. The fact that agitation dredging and underwater disposal of dredged material were conducted on so large a scale only made matters worse. Whereas some small, localized agitation operations might succeed in such an environment because of their relative size, the very scope of the Delaware dredging made it susceptible to the types of failure which occurred. Familiarity

with the Delaware and other large estuaries led Ippen (1966) to state in his classic text on estuarine and coastal processes:

> Dredging of channels should be accompanied by permanent removal of the sediments from the estuary. Dumping downstream is highly suspect and almost always useless. Agitation dredging falls into the same category, if permanent removal is desired.

Success in maintaining navigable depths in the Delaware was achieved only by removing dredged sediments from the estuarine system.

17. In light of the preceding paragraph, the Eastham Channel project (paragraph A2) might be considered as the exception that proves the rule. In this somewhat unusual case, the agitated material served a positive purpose by remaining within the local hydrodynamic system. When the material formerly agitated by hopper dredges was removed from the system by bucket dredges, the result was an alteration of local flow patterns and increased shoaling in navigation areas. While such a situation is not likely to be found often, it still points out the need for understanding the interaction between agitation dredging and the natural regime in which it is employed. In this case, the agitation dredging was successful in spite of initial ignorance concerning where the agitated material was going and what its effects were. However, that ignorance was later responsible for the harmful results of ceasing agitation dredging.

## Propwash

18. As with hopper dredges, propwash agitation dredging is composed of intentional and unintentional types. The unintentional type, caused by vessels moving through a waterway or freeing themselves after grounding, is often viewed as a problem. A brief description of this situation and some research into it is given in paragraphs A121 and A122. The intentional type of propwash agitation dredging may be conducted by a vessel especially designed or modified for that purpose, such as the *Salvage Chief* (paragraph A22) or the LCM (landing craft, mechanized) *Sandwick* (paragraph A36).

19. Propwash agitation dredging is described in Appendix A in varying degrees of detail for 13 different locations. At 12 of these locations, a vessel modified for the work was used. Of the 13 sites, 11 were in the Pacific Northwest. Natural regimes included coastal harbors, river mouths, river channels, and estuaries; and the material agitated was mostly sand.

20. Virtually every propwash agitation project in Appendix A was successful in the sense that some bottom material was moved. However, it becomes

apparent after reading about these projects that certain ones were more feasible than others. The highest reported dredging rate in a nonexperimental project was 640 cu yd/hr at The Dalles Dam, by the LCM *Sandwick*. The *Sandwick* also moved 415 cu yd/hr in the Umpqua River in deeper water with the aid of currents (see paragraphs A36-A42 for descriptions of these projects and similar ones). The *Salvage Chief*, a larger and far more powerful vessel than the *Sandwick*, achieved an average rate of 470 cu yd/hr in the Chinook Channel project (paragraphs A22-A30), which covered a fairly long dredging area.

21. At the opposite end of the feasibility spectrum were projects such as Mare Island Naval Shipyard, where a brief test of propwash from a conventional, nonmodified tug produced a significant turbidity plume but no measurable bottom changes (paragraph A102). Currents during the test were negligible. In both the Rogue River and Missouri River tests (paragraphs A36-A42 and A31-A45), mixed results were achieved, although the latter was primarily an experiment to test different operational modes of a modified LCM.

22. The average performance of vessels such as the *Sandwick* seems to lie in the range of 200 to 300 cu yd/hr in sand and more in finer material, depending on a number of conditions (see Table A2, page A49). Short-term excavation rates are usually much higher (150 cu yd were excavated in 2.5 min in the Tillamook Bay tests, paragraphs A43-A53), but a significant amount of time can be used in repositioning the vessel periodically. In addition, there is some evidence that propwash agitation dredging may increase vessel downtime due to repairs. Optimum water depths for propwash agitation dredging appear to be between two and three times the draft of the agitating vessel for sand. High excavation rates are possible in shallow water; but the depth of excavation may be excessive, and a berm of excavated material may form on the downstream side. Laboratory tests reported in the Appendix A section on propeller jet erosion (paragraphs A118-A122) suggest a maximum feasible depth of four times the agitating vessel's draft.

23. Propwash agitation dredging seems best suited at present for areas with little wave action. Waves caused anchoring problems when operating in the channel mouth at the Rogue River and Chetco River projects (paragraphs A36-A42). At the Rogue River, the *Sandwick* was limited to working in waves less than 2 ft high. Weather conditions also caused delays in the Tillamook Bay experiments.

24. Cementing, cohesion, or compaction of the bottom sediment can make

propwash agitation dredging difficult to perform. At the Baker Bay project (paragraphs A36-A42), partially cemented sand had to be loosened by dragging anchors across the shoal. Failure of the tug propwash to move appreciable amounts of material at Mare Island Naval Shipyard was blamed in part on consolidation of the fine bottom sediments.

25. Augmentation of the propwash current by natural currents was cited in several projects as being especially helpful. In the Umpqua River (paragraphs A36-A42), propwash agitation was successful in 20 ft of water (roughly 3.5 times the *Sandwick's* draft) largely because of existing currents of 2 to 4 fps. A respectable dredging rate of 265 cu yd/hr was achieved in the Cowlitz River project in coarse sand and gravel (paragraphs A36-A42) partly because of 3- to 5-fps natural currents. The Missouri River tests of a number of operational modes showed that the most effective by far was for the propwash craft to begin at the upstream side of a shoal and work downstream with the propwash directed downstream. Operating in concert with local currents is especially important in tidal areas, where agitation often is done only on ebb flow.

26. In summary, a technically feasible propwash agitation dredging project is likely to have a number of the following characteristics:

a. Propwash vessel fitted with adjustable deflector device and convenient anchoring system.

b. Localized, well-defined shoaling in moderate water depths.

c. Fine, noncohesive, uncompacted shoal material.

d. Natural currents that augment the agitation and transport processes.

e. Little or no wave action.

## Vertical mixers and air bubblers

27. Vertical mixers such as the Helixor and Ventra Vac units tested at Grays Harbor (paragraphs A74-A85) and Mare Island Naval Shipyard (paragraphs A112 and A113) and air bubblers such as the Harbour Town Marina unit (paragraphs A3-A11) and the model air curtain tested in the Mare Island laboratory studies (paragraph A90) are grouped together because they claim the same basic operating principle: by releasing compressed air near the bottom, the devices induce currents in the water column rising from the bottom to the surface. These currents are supposed to carry with them sediment from the bottom and near-bottom, at least part of which is to be resuspended by horizontal currents feeding the rising vertical currents.

28. In theory, such devices should work by maintaining sediment in suspension until natural currents can flush it away. In practice, however, no significant results were noted at any of the three field test sites reported in Appendix A, and only limited results were obtained from the laboratory study. There were a number of potential reasons for this; for instance, the Grays Harbor units may have been installed in conditions unfavorable to their operation. However, there appear to be some fundamental problems with how the operating principle of such devices relates to the results they try to achieve in agitation dredging.

29. There is no question that devices such as the Helixor, Ventra Vac, and air bubblers can induce significant rising vertical currents extending to the water surface. The air-lift principle has been used successfully for years in underwater mining (Clauss 1971), and numerous studies have been conducted on the structure of vertical currents from air bubblers (Wilkinson 1979, Cederwall and Ditmars 1970, Kobus 1968). Bubble screens have been used for a number of years to inhibit saline density currents from penetrating into harbors, slips, and locks (Abraham and Van den Burgh 1964, Rahm and Sjoberg 1965, Simmons 1967) and field tests have been performed on their ability to reduce sedimentation by acting as barriers to sediment-laden density currents (DeNekker and Knol 1968). The turbulence and horizontal currents produced at the surface by air bubblers have been used successfully to dissipate wave energy (Green 1961) and inhibit ice formation (Ashton 1974).

30. All of the above applications of air bubblers and vertical mixers are based primarily on the vertical currents generated. In agitation dredging with these devices, however, the horizontal flow patterns and flow velocities are of equal importance, since horizontal flow is what brings sediment to the vertical plume. Figure 3 shows a simplified velocity field for horizontal flow near an air bubble plume from a line source, schematized from Cederwall and Ditmars (1970) and Bulson (1961). Flow into the plume occurs over the lower three-fourths of the water column from both sides at an average velocity $V_I$. Assuming this flow is fully entrained in the plume, it rises into the upper one-fourth of the water column, where it disperses away from the plume with a velocity distribution approximately as shown. Maximum velocity $V_s$ occurs at the surface. Simple calculations show that for such a situation, $V_I$ would be on the order of one-sixth $V_s$. In a very powerful prototype installation using a jet engine as a compressor, Bulson measured a surface velocity

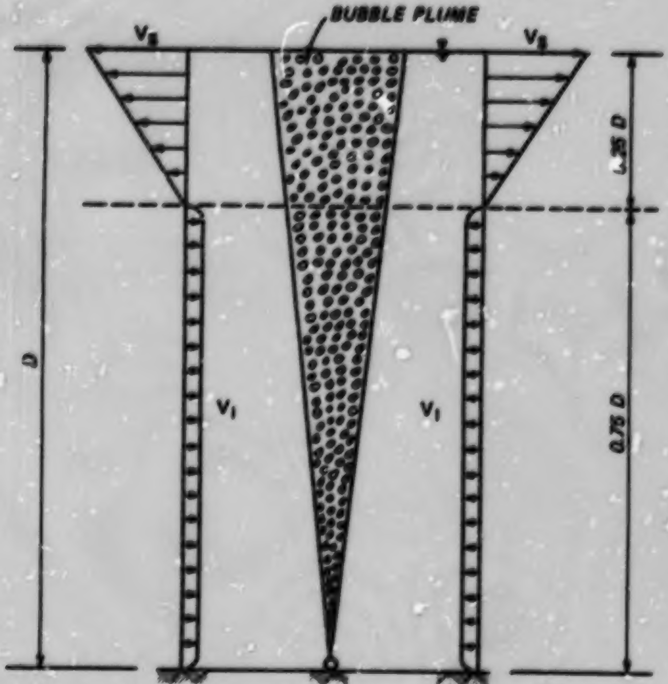

Figure 3. Schematized horizontal velocity field near line source air bubble plume

of 3.6 fps and an average velocity into the bubble plume of 0.6 fps in 25.5 ft of water. Bulson also showed that such velocities decay rapidly with distance from the bubble plume. His measured surface velocities became negligible at distances of eight times the water depth away from the plume for all cases except the shallowest water. If flow into the plume behaves in a similar manner, it will reach a negligible threshold even closer to the plume. Bulson also showed that the magnitude of $V_s$ varied as the cube root of airflow supplied to the bubbler in standard units. In other words, to double $V_s$ and $V_I$ would require that airflow (and power) be increased eight times.

31. To summarize the above discussion, other investigators have shown that:

- a. Horizontal currents feeding into line source air bubblers are relatively weak, even in installations with large amounts of power.
- b. The zone of influence of such currents is limited.
- c. Exponential power increases are needed to increase horizontal flow into the bubble plume.

32. An obvious conclusion is that the sediment resuspending potential of flow into the bubble plume is extremely limited, and that most sediment carried into the plume would have to be in suspension already. The Helixor and Ventra Vac units (Figures A53 and A55, pages A55 and A57) address this situation somewhat by making all flow into the units enter near the bottom, thus increasing flow scour potential near the units. This effect should be localized, however, as the flow streamlines would spread rapidly with distance from the unit. The Mare Island Naval Shipyard tests indicate that such a

situation actually occurs, since scour caused by the Ventra Vac units was limited to a small area around each unit's base.

33. Another possibility is that the water in the lower part of the water column will be denser than upper layers, due to greater salinity and/or lesser temperature. Such a situation often occurs in estuaries, where much of the past agitation dredging has been done. When this denser water is raised to the surface by vertical mixers or air bubblers, it is carried horizontally a limited distance by the surface currents, and then begins to sink back toward the bottom. As it sinks, part of the denser water mixes with the lighter upper layers, and part returns to the bottom to be drawn back into the vertical mixer or bubble plume along with water entrained from both the upper and lower layers. Thus a semiclosed loop is formed similar to that observed in the Grays Harbor tests. McAnally (1973) described such a pattern in laboratory tests of air bubblers in two-layer density-stratified water. The circulation and mixing patterns he observed are schematized in Figure 4. This phenomenon would further diminish the effectiveness of vertical mixers or air bubblers as agitation dredging devices, since part of the flow and sediment brought to the surface would be cycled back through the vertical flow pattern.

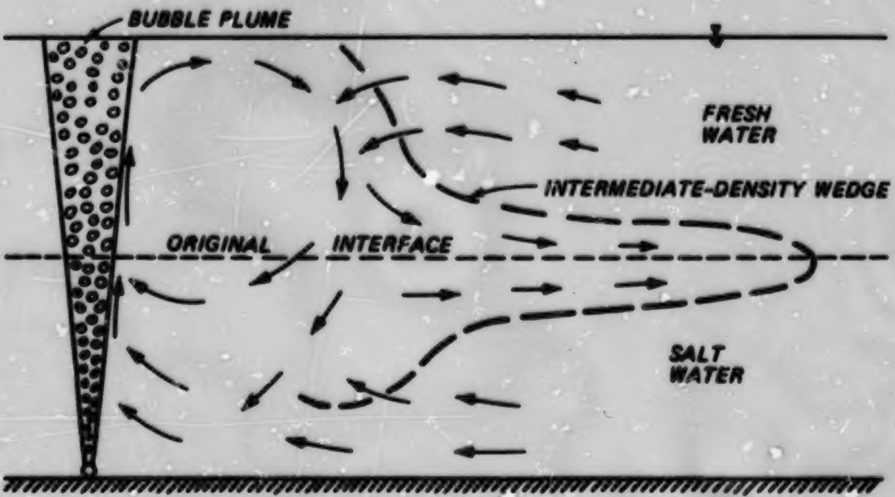

Figure 4. Flow pattern caused by air bubbler in two-layer, density-stratified water (schematized)

34. The preceding discussions suggest that the premise behind using air bubblers and vertical mixers as stationary agitation dredging devices has some fundamental faults. However, using such devices in a moving mode in

conjunction with mechanical agitation by rakes, drag beams, or other implements shows some promise. Luo and Gu (1981) reported significant increases in dredging efficiency using an "aerated rake" rather than a conventional one in both laboratory and field tests in the People's Republic of China (paragraphs A123-A126). In the Harbour Town Marina tests, the only noteworthy agitation occurred when the air bubbler was pulled along the harbor bottom while operating. The combination of air bubbler and mechanical agitation utilizes the best features of each device: the air bubbler's ability to produce vertical currents and the rake or drag beam's capacity to initially agitate even consolidated bottom material in the immediate vicinity of the bubble plume.

## Rakes and drag beams

35. Rakes, drag beams, and similar devices work by being pulled over the bottom, mechanically loosening the bottom material and raising it slightly in the water column. Although crude, they can be effective in areas with cemented, cohesive, or consolidated sediments; and they require no special equipment other than a vessel to pull them. In shallower water and with a large enough vessel, propwash may help in the agitation process as well. The draghead of a trailing suction hopper dredge acts as a rake to some degree as it is pulled along the bottom, since not all of the material it loosens is drawn into the suction tube.

36. Since rakes and drag beams produce no currents of their own and since they do not resuspend material as much as loosen it, they must be used in conjunction with natural currents strong enough to transport the loosened material away from the shoaling site. In the People's Republic of China, rake dragging was used in conjunction with currents created by releasing water from tide gates (paragraphs A123-A126). The bottom material was generally finer than 0.1 mm. In Savannah Harbor, silt-size material is regularly removed from slip areas using a drag beam, even though currents in the slips are weak (paragraphs A54-A75). Interestingly, both projects experienced difficulty in agitating compacted fine-grained material.

37. One possible way for helping the dragging process by the use of air bubblers is discussed in paragraph 34. Another way would be to deflect the propwash of the towing vessel downward toward the dragging device. A combination of the three--dragging, air bubbler, and propwash--might prove the most effective of all, especially when the towing vessel moves into a current so maximum use is made of the propwash.

38. Agitation dredging rates by rakes or drag beams vary considerably but can be quite high. The highest rate recorded in tests in Savannah Harbor was 3,320 cu yd/hr for a rather short time period. Long-term rates undoubtedly would be lower. The lowest rate in the Savannah Harbor tests was 240 cu yd/hr in compacted material sloping tranversely to the direction of dragging. The People's Republic of China data showed a definite correlation between dragging speed and dredging rate, with the maximum dredging rate occurring at a dragging speed of about 2 fps. The investigators also reported using different rake forms for different soil types.

### Water jets

39. Water jets for agitation dredging operate on the same fundamental principle as propwash agitation. The main differences are: (a) water jets can be grouped in any arrangement desired, (b) streams issuing from the jets usually originate close to or on the bottom rather than the surface, (c) water jets are usually used in a fixed location, and (d) water jets are usually intended for frequent operation to prevent large shoaling accumulations, whereas propwash is a remedial measure to remove shoal deposits. Because of the last point mentioned, water jet installations lend themselves to automatic operation. They may also have difficulty removing larger amounts of shoaling that might accumulate during periods of nonoperation.

40. Only one project described in Appendix A, the Mare Island Naval Shipyard, utilized water jets for agitation dredging (paragraphs A86-A117). However, several configurations were tested, and a good deal of laboratory work was performed to define the jets' operating characteristics. The project is also an excellent example of the comprehensive sedimentary and hydrodynamic analyses that should be performed prior to any type of agitation dredging, and especially before designing a fixed installation such as a water jet system. The mechanisms of sediment transport into the problem area were studied and related to hydrologic, tidal, and wave conditions; local hydrodynamics were measured; and the properties of shoaling material before and after deposition were investigated. The general sedimentary environment of Mare Island Strait was characterized, which is an important step toward choosing suitable methods of agitation dredging. Monitoring of local conditions during the test periods was thorough.

41. Two types of water jet arrays were field-tested at Mare Island Naval Shipyard. The first type, consisting of 70 jets located on the bottom

of a docking slip, was intended to remove newly deposited shoaling material before it had a chance to consolidate and gain shear strength. Although test results were encouraging, they could not be regarded as conclusive. The test period followed a 2-year winter drought during which little new sediment was introduced to the test area. Although no shoaling occurred over the water jet array while a nearby control area experienced almost 2 ft of accumulation, the test conditions were not representative of normal shoaling rates in the area.

42. The second type of water jet array consisted initially of 25 jets installed along a berth wall and angled downward to scour sediment from underneath docked ships. Initial results indicated that the jets had some effect on shoaling but were unable to completely prevent it. The jet array was modified to a line of 10 more powerful jets and operated for almost 2 years. A number of operational interruptions occurred during this test period, making analysis of the array's success difficult. Shoaling was not stopped totally, although there was some evidence that it was reduced. The effects of each jet on the bottom showed an apparent sensitivity to the jet's downward deflection angle, although other factors could have been involved. As with the first water jet array, results were encouraging but not conclusive.

43. Another application of water jets similar to agitation dredging was described by Ali and Halliwell (1980). They conducted a series of laboratory studies into the general properties of water jets for scouring and applied the results to model tests of a particular problem. The problem was to increase the scouring potential of discharge from a particular lock sluice by modifying the end of the sluice into a jet. By working with two different scale models, they arrived at a design consisting of a prefabricated circular nozzle with an opening 3.14 ft in diameter (prototype). Model tests of this nozzle installed horizontally in a mock-up of the actual lock entrance showed that it significantly increased near-bottom velocities of the lock discharge. Hopefully, this increase would be sufficient to clear the lock entrance of shoaling material accumulated between discharges; however, no mention was made in the reference of prototype applications. This type of operation is not classified as agitation dredging because most of the "equipment" used was intended primarily for other functions. The methods employed were similar in concept but not in scale to the Mare Island Naval Shipyard tests.

## Environmental

44. Table 1 shows that some degree of environmental monitoring was conducted at six of the twenty-two example agitation dredging projects in Appendix A. Of those six, four were in fine-grained sediment and the others in sand. Four projects were located in estuaries, one in a saltwater sound, and one in a Gulf of Mexico approach channel. Two of the projects were performed by propwash agitation, and one each was performed by air bubbler, drag beam, vertical mixer, and hopper dredge. Therefore, despite a relatively small sample, a good range of environment and sediment types and a wide range of equipment types were covered by this monitoring.

45. Three of the projects described in Appendix A were monitored fairly completely for environmental effects of agitation dredging: Chinook Channel (paragraphs A22-A30), Tillamook Bay (paragraphs A43-A53), and Savannah Harbor (paragraphs A54-A73). At these three sites, biological as well as water quality data were collected. At the Calcasieu River project (paragraphs A127-A143), water quality was analyzed thoroughly but no biological samples were taken. Grays Harbor (paragraphs A74-A85) was monitored for suspended sediments, dissolved oxygen, temperature, and salinity, while the Harbour Town Marina results are reported only in terms of turbidity (paragraphs A3-A11). In keeping with the format of the previous section on technical feasibility, environmental monitoring results will be discussed in terms of the equipment or method used.

### Hopper dredges

46. In the Calcasieu River approach channel, the following changes in water quality were noted in the immediate vicinity of hopper dredge agitation:

a. Temperature, pH, conductivity, and dissolved oxygen were virtually unchanged.

b. Significant increases in arsenic, chromium, copper, lead, nickel, zinc, total organic carbon, and suspended solids in the water column occurred during agitation dredging near middepth in the water column. These increases disappeared soon after dredging ceased.

c. Water quality was generally much poorer in the dredge hopper than in the water column outside the dredge.

### Propwash

47. The following is a summary of environmental effects noted during the Chinook Channel propwash tests:

- a. Agitation turbidity plumes temporarily lowered dissolved oxygen to marginal levels.
- b. Benthic organisms decreased both in the agitation dredging area and downstream, but had recovered 6 months later.
- c. Diversity and richness of epibenthic and demersal organisms were lowered during agitation dredging, but were back to normal within 2 weeks.
- d. Water quality remained biologically acceptable during and after agitation dredging.

48. Because of inclement weather and high freshwater inflows during the Tillamook Bay propwash tests, it was difficult to separate natural environmental changes from those caused by agitation dredging. The following points were noted:

- a. No measurable water quality changes occurred that could be related to sediment resuspension.
- b. No acute toxicity or sedimentation effects were observed on organisms near the dredging site.

## Vertical mixers and air bubblers

49. The primary environmental effects measured in this category of equipment were in the Grays Harbor tests of vertical mixers:

- a. Dissolved oxygen was increased 18 percent in the vicinity of the mixers.
- b. Suspended sediment was increased somewhat near the mixers.
- c. Density-related water stratifications were disrupted within 20 ft of the mixers.

## Rakes and drag beams

50. The Savannah Harbor environmental monitoring was one of the most complete reported in Appendix A. An interesting aspect of this monitoring is that the sediments agitated were polluted with oxygen-demanding organics, grease and oil, and Kjeldahl nitrogen. The following points were made regarding drag beam agitation in this polluted sediment:

- a. Suspended solids and turbidity were increased by as much as an order of magnitude in the lower part of the water column. The amount of this increase varied with dredging rate and distance from the dredging site.
- b. The level of suspended solids and turbidity occasionally increased with time during agitation operations.
- c. Changes in dissolved oxygen, COD, and ammonia nitrogen and transfer of sediment pollutants to the water column were small

and varied, so that no trends could be established separating agitation dredging effects from naturally induced variations.

d. Dredged areas were lacking in benthic organisms prior to testing because of natural or agitation dredging effects or both.

Water jets

51. No environmental monitoring relative to water jets was reported in the projects listed in Appendix A.

Summary

52. Several hypotheses about the environmental effects of agitation dredging can be made based on the examples described in this section:

a. The environmental effects of agitation dredging are often of the same order of magnitude or less than naturally occurring variations, making it difficult to separate the two.

b. When water quality effects of agitation dredging are measurable, they are usually confined to the region near the dredging site. In past projects, these effects have usually diminished rapidly following agitation dredging.

c. Agitation dredging methods that raise sediment high in the water column probably affect water quality more than methods that achieve similar dredging results by raising the material a limited distance.

d. The longest lasting biological effect of agitation dredging may be the disruption of benthic organisms in the dredging and downstream areas. Recovery of these organisms may be slower at the dredging site than in surrounding areas.

e. Frequent agitation dredging may cause a permanent disruption of benthic life.

53. As mentioned earlier, the adverse perception of agitation dredging is sometimes more of a problem than its actual environmental effects. Turbidity and its associated potentials for reduced water quality, smothering of shellfish beds, etc., traditionally have been primary environmental objections to dredging of any type. Since agitation dredging by definition must raise suspended sediment concentrations temporarily, the image it creates in this respect is especially strong. This unfavorable perception is further reinforced when visible turbidity plumes are created by agitation dredging.

54. The actual environmental effects of agitation dredging may be reduced by investigations prior to dredging. Determining the nature and pollutant content of the sediment to be agitated, surveying the surrounding region for sensitive sites such as spawning areas and shellfish beds, and predicting deposition sites for the agitated sediment are examples of such investigations.

Numerical modeling of agitated sediment transport may be advisable in some cases. The perceived environmental effects of agitation dredging may also be reduced by the information gained from a thorough predredging investigation, as well as by employing operational techniques that minimize visible turbidity. Such techniques include: (a) raising sediment in the water column only as far as needed to accomplish removal from the site, (b) scheduling agitation dredging to coincide with stronger currents so that turbidity is dispersed as rapidly as possible, and (c) working during periods of high natural turbidity so that the increase due to agitation dredging is proportionately less. These techniques may also lessen the actual environmental effects of an agitation dredging operation.

## Economic

55. The most meaningful way of determining the economic feasibility of different dredging methods would be to compare their costs to achieve comparable results under identical conditions in a number of different situations (the term "comparable results" is especially important in the case of agitation dredging, where published cost figures may include material that later returned to the dredging area). Obviously, such circumstances do not exist for the agitation dredging methods discussed in this report, nor are they likely to ever occur. The best that can be done is to analyze the available cost data to see if patterns regarding relative economic feasibility can be identified.

56. Seven of the example projects in Appendix A present some economic data. Every method of agitation dredging discussed in Appendix A except vertical mixers and air bubblers is covered to some degree. Therefore, as in the previous sections on technical and environmental feasibility, discussions of economic feasibility are presented by the type of method or equipment used.

### Hopper dredges

57. The most extensive cost data in Appendix A were for the hopper dredge *Langfitt* operating in the Louisiana Gulf Coast area (paragraphs A127-A143). These data showed that the *Langfitt* was several times cheaper and several times more productive in the agitation mode as opposed to hauling and dumping. The cost per cubic yard for the *Langfitt* was less in either mode than for any of the selected dredges listed in Table A7 (page A97). The

average cost per cubic yard for the *Langfitt* in the years 1965-1972 was $0.062 for agitation maintenance and $0.210 for hauling and dumping. During this period, the average distance to the *Langfitt's* dump area was slightly less than 2 miles, which is relatively short.

58. Figure 5 shows a comparison of the *Langfitt's* cost per cubic yard in the maintenance agitation and hauling modes with the average costs of all Corps hopper dredged in the same modes, weighted according to the total cubic yards produced by each dredge in each mode in a given year. Since the *Langfitt* produced most of the agitation maintenance volumes in the period covered by Figure 5, the agitation dredging plots for it and for all hopper dredges are virtually identical, even though a comparison of Tables A6 and A7 (page A97) shows that the other dredges had much higher unit agitation costs.

59. The differences between hauling and agitating costs for all hopper dredges are dramatic and apparently increasing. The hauling costs increased at an average rate of $0.025 per cubic yard per year, while the agitation costs increased only $0.003 per cubic yard per year, one-eighth as much. The *Langfitt's* hauling costs varied considerably, but still increased at an

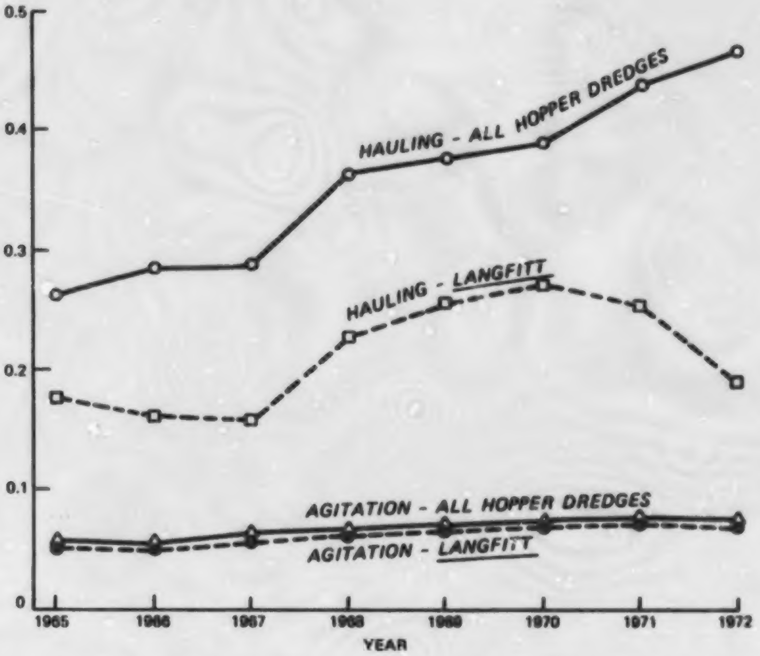

Figure 5. Maintenance dredging costs for all Corps of Engineers hopper dredges and hopper dredge *Langfitt*, 1965-1972

average rate of $0.011 per cubic yard per year based on a linear regression of the data in Figure 5. By 1972, the weighted hauling cost per cubic yard for all hopper dredges was 6.2 times that for agitation. The magnitudes of all these differences are probably due in part to the predominance of the *Langfitt's* operations in figuring weighted agitation costs, but there can be little question that the trends represented are correct.

60. In the Delaware Estuary (paragraphs A12-A21), where hopper dredge agitation was found to be a major cause of high shoaling rates, the relationships found for the Louisiana Gulf Coast projects also seemed to apply. The costs per cubic yard for agitation dredging were less than those for hauling, and the apparent production rates were higher. The key word is "apparent," because the agitated material often resettled in the navigation channel in a less dense form, actually increasing the effective shoaling volumes. Once agitation dredging was stopped and all dredged material removed from the estuary, the total cost for maintaining the estuary decreased by 67 percent, even though dredging unit costs increased 53 percent.

61. The Delaware Estuary example points out the reason why "removal of bottom material from a selected area" is a key phrase in the definition of agitation dredging offered at the beginning of this report. The failure of agitation dredging to accomplish this removal in the Delaware Estuary completely negated what appeared to be significant cost advantages. In turn, this failure was traced directly to an insufficient understanding of local hydrodynamics and sediment transport patterns. In other words, agitation dredging in the Delaware Estuary was technically infeasible which therefore made it economically infeasible compared with methods used later.

## Propwash

62. The Chinook Channel project (paragraphs A22-A30) and five of the nine projects under the heading of Pacific Northwest (paragraphs A36-A42) present some cost data. The average cost per cubic yard in the Chinook Channel work was $2.80, while the Pacific Northwest costs ranged from $0.24 to $0.63 per cubic yard (see Table A2, page A32). No comparison was made in the Chinook Channel report with costs for other dredging methods. However, in the Pacific Northwest projects, propwash agitation dredging costs were 40 to 90 percent less per cubic yard than costs for other methods used previously at the same sites. The relatively high cost per cubic yard for the Chinook Channel project may have been due in part to the experimental nature of work there and to

the larger and more powerful vessel used. The only other pattern that can be identified is that in the Pacific Northwest tests, the costs per cubic yard in silt and fine sand were less than half of those in sand or sand and gravel. This was true even though predredging water depths were substantially greater at the sites with silt and fine sand, and other factors such as currents, volumes dredged, and wave action were of comparable magnitudes.

### Vertical mixers and air bubblers

63. No cost data are given in Appendix A for vertical mixers or air bubblers (paragraphs A3-A11, A74-A85, and A112 and A113).

### Rakes and drag beams

64. The Savannah Harbor project described in Appendix A (paragraphs A54-A73) contains a substantial amount of data on drag beam agitation dredging costs and physical parameters relating to these costs, summarized in Table A3 (page A41). At first glance, the cost data might appear suspect because of their relative uniformity from site to site. The range of costs is \$0.076 to \$0.25 per cubic yard, with most sites reported as \$0.13. However, there are variations in the cost data which together with other data given in Table A4 (page A45) correlate with reasonable assumptions that can be made about this type of agitation dredging. This correlation is explained in the following paragraphs.

65. Stuber (1976) lists rental rates for two tugboats used in the Savannah River drag beam operations. The tugboat used for the Colonial Oil location rented for \$50/hr, while the one used at the Georgia Ports Authority Ocean Terminal Slip No. 1 and ITC, Inc., cost \$87.50/hr. In addition, the Savannah District of the US Army Corps of Engineers assessed a charge at the time of the Savannah Harbor study of \$176/hr of agitation dredging. Assuming that agitation dredging by drag beam is charged on an hourly basis, the relationship between hourly charge, $C$, average removal rate, $R$, and unit cost, $X$ would be:

$$X = \frac{C}{R} \qquad (1)$$

This is not an unreasonable assumption, since fixed costs such as mobilization and demobilization would be negligible for the types of operations conducted in Savannah Harbor.

66. Substituting into Equation 1 an average hourly charge of \$245, the

relationship between X and R would be as shown in Figure 6. This value for C was obtained by adding the average of the two tugboat charges given by Stuber to the hourly charge assessed by the Savannah District. Using the root-mean-square of the average removal rates given in Table A4 as a representative removal rate for each site, the unit costs given in Table A3 for the sites listed in Table A4 can be plotted in Figure 6.

67. The points plotted in Figure 6 do not fit the hypothetical curve exactly, nor should they be expected to considering the diverse and approximate natures of both plots. However, they do follow the general trend expressed by the curve, indicating that the cost data in Table A3 may be relatively accurate.

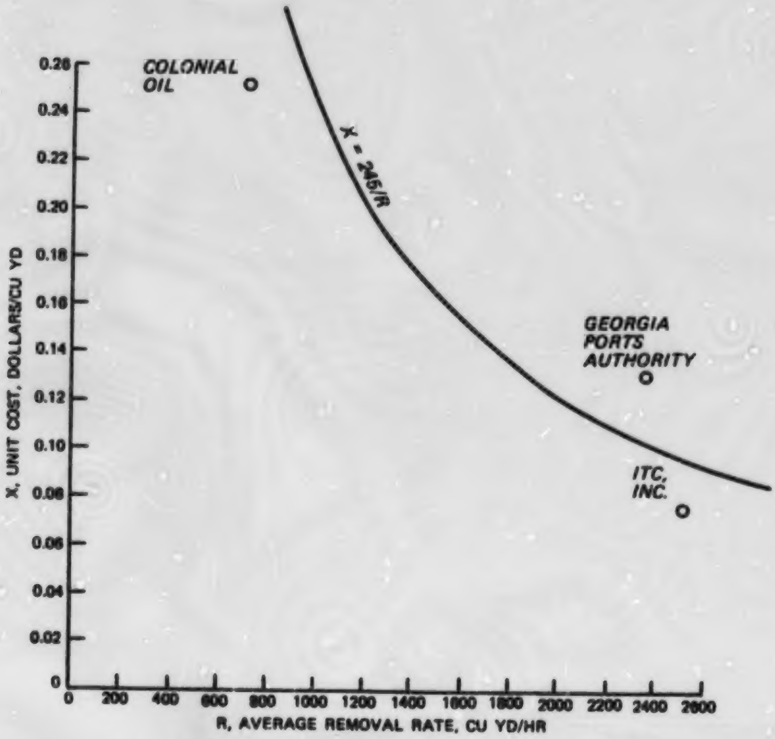

Figure 6. Average removal rate versus unit dredging cost, Savannah Harbor

68. Unit costs for drag beam agitation in Savannah Harbor show no clear-cut dependence on the physical parameters given in Table A3. No patterns of cost are apparent relative to water depth, berth type, berth dimensions, frequency of dredging, amount or depth of excavation. The one pattern that does appear is that much more material is removed from most of the slips listed

than from any of the parallel berths. Several possible explanations may be made; the most likely is that the slips trap large amounts of material because of their configuration and that the material settles in relatively quiescent conditions, making it easier to remove by frequent agitation.

69. In the People's Republic of China projects (paragraphs A123-A126), one rake dragging operation was estimated to have accomplished sediment removal at a cost 10 percent that of conventional dredging. This is well in line with the unit costs reported in Savannah Harbor, as Hussey, Gay, and Bell, Inc. (1975) estimated that hydraulic pipeline or clamshell dredging would be 30 times more expensive there than the drag beam methods presently used. Tests with aerated rakes at another location in China resulted in unit costs that were one-third those of conventional rakes.

Water jets

70. The situation for determining the economic feasibility of water jets as agitation dredging devices is only slightly better than that for vertical mixers and air bubblers. Indications are that water jets such as those tested at Mare Island Naval Shipyard (paragraphs A86-A117) can be at least partially effective in removing recently settled sediment, but operational and other problems have thus far prevented a definitive demonstration of their range of technical feasibility. Without reliable numbers for the design parameters producing this range, an economic analysis such as that given by Bailard (1980) is limited to defining some of the bounds of economic feasibility. For the example given by Bailard, a water jet array with a design bottom stress level of $8.3 \times 10^{-3}$ lb/ft$^2$, jet duty cycle of 12 min, and pipe water velocity of 8.2 fps maximum would become economically feasible only when dredging costs reached $6.00/sq yd of bottom. For a 1.5-ft average excavation, such as that accomplished by the Berth 7 jet array under comparable conditions at Mare Island Naval Shipyard, dredging costs would have to reach $12.00/cu yd for a jet array to be economically feasible. However, as Bailard points out, such an analysis does not take into account the additional benefits offered by a technically feasible water jet system, such as not having to move moored ships for dredging operations and having slips maintained continuously at or near design depths.

## PART III: SUMMARY

### Conclusions

71. Several general conclusions about agitation dredging can be made based on the example projects discussed in this report:

- a. Agitation dredging can perform some of the same maintenance functions as conventional dredging equipment, given the proper conditions.
- b. Correctly applied, agitation dredging can be conducted with minimal effects on the environment.
- c. Reported agitation dredging costs are almost always less than those for conventional dredging.
- d. Taking a, b, and c into account, agitation dredging may be underused as a maintenance dredging technique, at least in the United States.

### Conditional

72. Some conclusions about the types of conditions leading to successful agitation dredging projects also can be reached:

- a. Agitation dredging generally works best in fine-grained, recently deposited material. As grain sizes increase and/or resistance to resuspension builds due to consolidation, cohesion, or cementation, the excavating efficiency of many agitation dredging methods begins to drop.
- b. Agitation dredging is used best for alleviating specific, well-defined shoaling problems. It is not well suited for general maintenance dredging over a large area.
- c. For many situations, agitation dredging could be used as a supplement to conventional dredging techniques. It can reduce the frequency of conventional dredging in problem areas and might be used to increase the efficiency of conventional dredging by moving material into central areas.

### Specific

73. Several specific conclusions are apparent from the Appendix A example projects:

- a. Of the agitation dredging methods presented in this report, the three that appear most feasible technically are:
  - (1) Hopper dredge overflow.
  - (2) Propwash.
  - (3) Rake or beam dragging.

b. Hopper dredge agitation can allow a project to be maintained with a relatively small dredge compared with the size dredge required for hauling and dumping.

c. Hopper dredge agitation costs are several times less per cubic yard than hopper dredge hauling costs, and may be increasing at a slower rate.

d. Propwash agitation works best in water with moderate depths (two to three times the propwash vessel draft) and little wave action, using a vessel especially modified for the task. Average removal rates are 200 to 300 cu yd/hr in sand.

e. Propwash agitation costs were reported as being several times less than costs for conventional dredging equipment performing the same work.

f. Rake or beam dragging agitation can achieve very high removal rates under favorable conditions. Removal rates appear very sensitive to the degree of compaction of bottom material.

g. Rake or beam dragging may be the least expensive agitation dredging method studied. Reported costs per cubic yard ranged from 1/10 to 1/30 of estimated costs for conventional dredging.

h. Based on published data, none of the three most feasible agitation dredging methods--hopper dredge overflow, propwash, or rake or beam dragging--can be identified as conclusively better or worse from an environmental standpoint than the other two.

i. Frequency of agitation dredging at a particular site and the height to which material is raised in the water column may be important factors in determining, respectively, the long- and short-term environmental effects of agitation dredging.

## Recommendations

74. One of the general conclusions drawn from this report is that agitation dredging may be underused as a maintenance technique, given its demonstrated advantages and potential for minimal environmental effects. Several reasons for this situation can be postulated:

a. Unfamiliarity with its uses. Agitation dredging simply may not be thought of when compiling dredging alternatives for a maintenance project.

b. Unfavorable image. The adverse perception of agitation dredging was discussed in the section on environmental feasibility. This perception alone often may cause selection of less controversial dredging methods, even though they may be several times more expensive.

c. Lack of standard, predictable technology. Except for the hopper dredge projects, every agitation dredging example discussed in Appendix A used "one-of-a-kind" equipment. This equipment was either: (1) experimental, (2) designed especially for one project, or (3) used under a relatively narrow range of conditions at different locations. Therefore, even if a project manager wanted to consider agitation dredging and was willing to cope with potential image problems, it would be impossible to find readily available equipment or to predict how that equipment might perform. Also, it would be very difficult to estimate the environmental effects of the operations.

d. Undeveloped state of the art. A corollary of the situation described in c is that most agitation dredging projects, even the successful ones, are accomplished using rather crude, brute-force equipment. Little effort has been expended toward developing and refining even proven methods such as hopper overflow, propwash, and rake and beam dragging. As conventional dredging equipment responds to the need for improved efficiency, existing agitation dredging methods appear left over from another era.

e. Need for hydrodynamic and sediment transport analyses. As pointed out and shown by example many times in this report, the hydrodynamic and sediment transport regimes at a project site are integral parts of any agitation dredging operation. Therefore, in order to conduct agitation dredging, the project manager is forced to deal with these subjects in much greater detail than if conventional dredging equipment were used. Data may have to be collected over long time intervals and analyzed by persons not usually involved in dredging operations. Although the major portions of such analyses should have to be done only once for a given project, and although the total cost of data collection, analysis, and agitation dredging may still be less than conventional dredging, the uniqueness of this requirement may work against agitation dredging.

75. All of the above reasons why agitation dredging is not used more point in one direction--a potentially viable but undeveloped technology. Although conventional dredging in the United States often has been accused of being technically underdeveloped, it is advanced compared with agitation dredging. This situation is essentially unnecessary, since all of the reasons presented in paragraph 74 can be dealt with using existing knowledge and a small amount of research. As discussed in the Appendix A section on propeller jet erosion, the information for improving agitation dredging methods is often available in a somewhat different but still applicable form.

76. The recommendation resulting from this report, therefore, is that the foundation established by it be built upon to develop agitation dredging as a usable alternative and supplement to conventional dredging methods. This development should focus on two areas:

a. Improving and standardizing the technology.

b. Synthesizing a unified site-specific approach to planning and conducting agitation dredging.

77. The result of such development would be to add a potentially cost-effective dredging method to the list of navigation channel maintenance techniques. As the costs of conventional dredging methods continue to rise, the possible benefits of a workable agitation dredging technology should not be ignored.

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# APPENDIX A: EXAMPLE AGITATION DREDGING PROJECTS AND EXPERIMENTS

1. This appendix contains descriptions of 22 agitation dredging projects and experiments covering a wide range of equipment, techniques, conditions, and results. Descriptions are drawn from literature on the projects or experiments and rely primarily on this literature for their accuracy. Since the hydrodynamic and sedimentary regimes are very important in agitation dredging, they are described in detail wherever possible. The projects and experiments covered in this appendix are the basis for conclusions and recommendations presented in the main text.

## Eastham Channel

2. Eastham Channel is located in the Mersey Estuary near Liverpool, England (Figure A1). Hammond (1969)* describes dredging operations in the channel and shows how the large amount of agitation associated with one type of dredging was actually beneficial to the area. Figure A2 shows the pattern

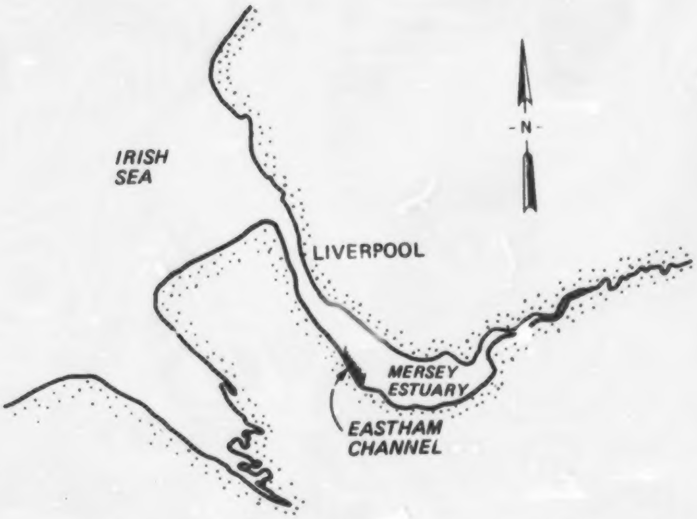

Figure A1. Location map, Eastham Channel

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* References cited in this appendix are included in the References at the end of the main text.

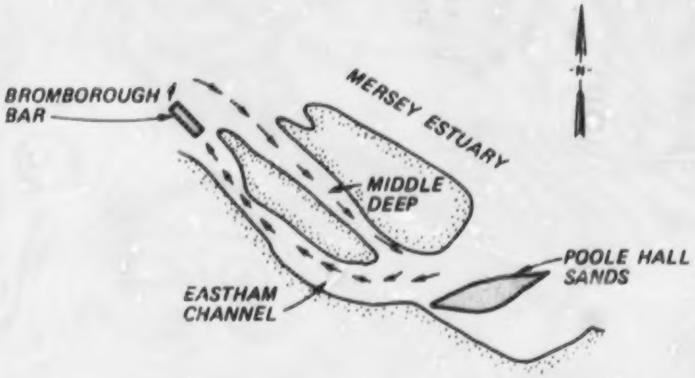

Figure A2. Net circulation, Eastham Channel area (from Hammond 1969)

of net water and sediment flow in the vicinity of Eastham Channel. Trailing suction dredging in the area of Bromborough Bar operated largely by agitation, with roughly twice as much material being moved from the bar due to draghead agitation and hopper overflow as was carried away in the hopper and dumped. Therefore only one-third of the material moved by the trailing suction dredge in maintaining depths at Bromborough Bar was actually removed from the semi-closed circulation system shown in Figure A2. Much of the agitated material was carried by currents into Middle Deep Channel, maintaining it in a partially shoaled condition. When attempts were made to deepen Bromborough Bar in the period 1952-1962, large quantities of material were removed from the entire circulation system by bucket dredges, which also caused little agitation. The result was that Middle Deep Channel widened and deepened due to a lack of agitatied material from Bromborough Bar. Flood tidal currents became stronger through Middle Deep, removing large amounts of material from Poole Hall Sands. This material was then moved through Eastham Channel and settled out at Bromborough Bar. Therefore the effect of bucket dredging at Bromborough Bar was to actually worsen conditions at the bar and in Eastham Channel. The recommended solution to the problem was to decrease the quantity of material removed by dredging from the circulation system. This was done by returning to trailing suction dredges to maintain Bromborough Bar.

## Harbour Town Marina

3. Harbour Town Marina is a small-craft harbor located on Hilton Head Island, South Carolina (Figure A3). The harbor consists of segments of two concentric circles connected to Calibogue Sound by a 400-ft-long, funnel-shaped channel (Figure A4). Mean tide range is 7.2 ft, and the surface area of the area of the marina is roughly 24,000 sq ft. Design depth in the harbor channel is 12 ft at mean high water. Maximum flood and ebb currents in Calibogue Sound are 1.2 and 2.0 fps, respectively, while calculated tidal currents in the Harbour Town Marina channel are several orders of magnitude less. The harbor and channel have significant shoaling problems, particularly adjacent to sheet-pile walls. These walls line the harbor and extend along roughly 150 ft of the channel. The remainder of the channel out to Calibogue Sound is bordered by marsh and mud flats in the intertidal zone. These flats may serve as a source of shoaling material for the marina when sediment is

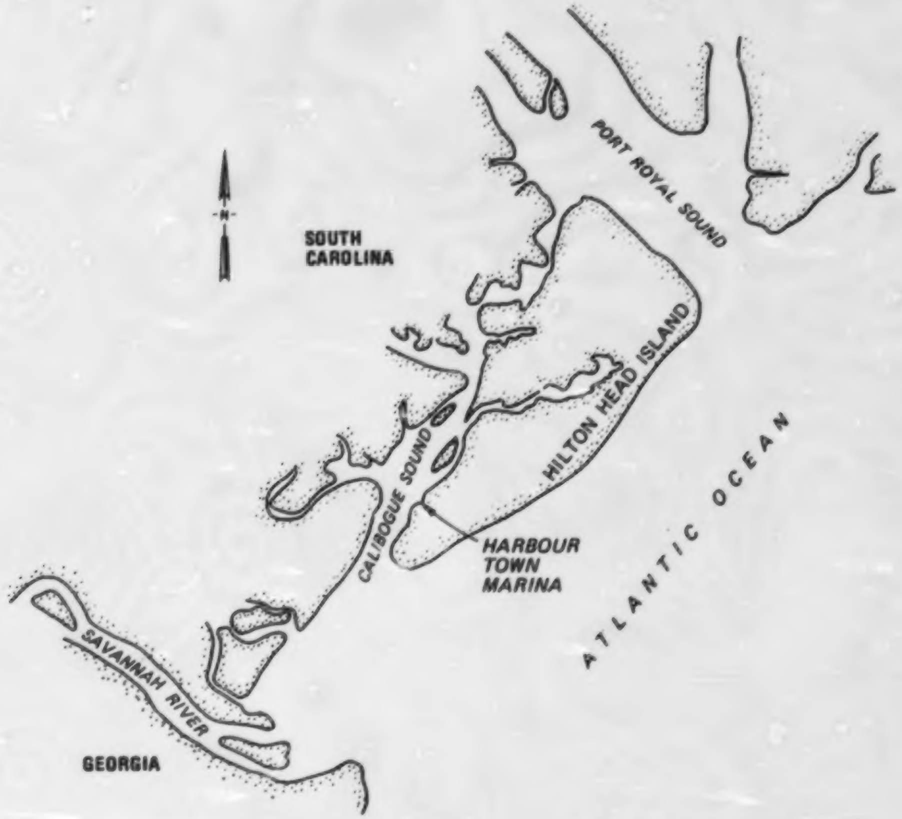
N
SOUTH CAROLINA
PORT ROYAL SOUND
HILTON HEAD ISLAND
CALIBOGUE SOUND
HARBOUR TOWN MARINA
ATLANTIC OCEAN
SAVANNAH RIVER
GEORGIA

Figure A3. Location map, Harbour Town Marina

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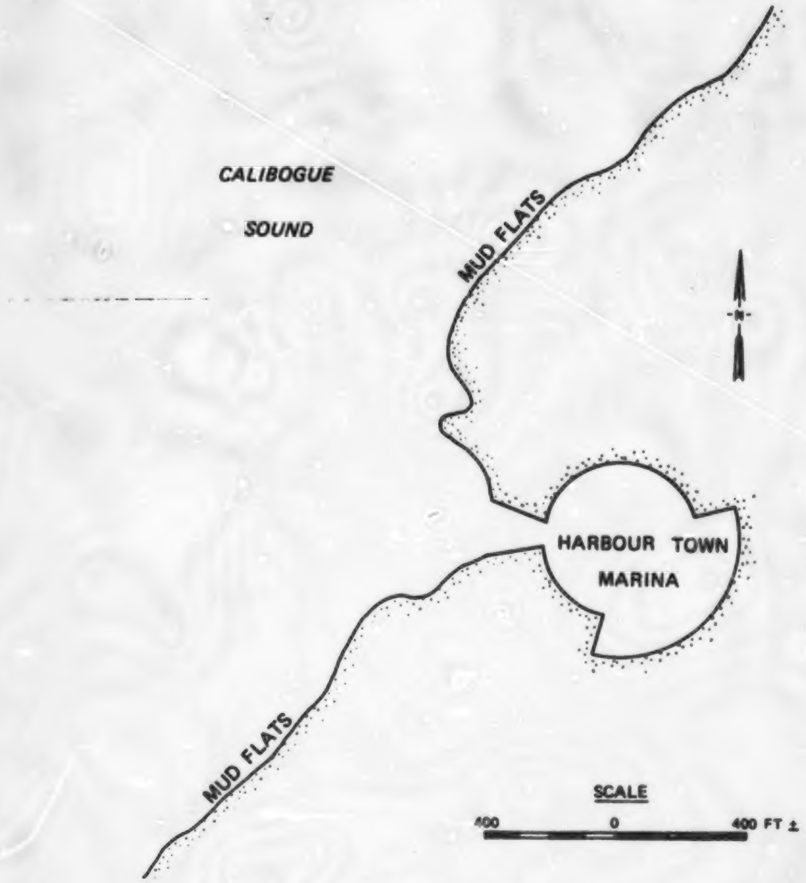

Figure A4. Plan view, Harbour Town Marina

suspended by wave action and moved by tidal or wind-generated currents. Therefore, in order for agitation dredging to work in Harbour Town Marina, material agitated in the harbor would have to be carried far enough into Calibogue Sound during an ebb tide that most of it would not be carried back on the subsequent flood. Very roughly, this would mean a transport distance of 700 to 800 ft from the center of the marina.

4. An agitation device was tested at Harbour Town Marina in March and April 1980. The intent of the device was to raise shoaled material high enough in the water column during an ebb tide that the weak ebb currents could carry it the required distance out of the harbor. Success of the device as the main component of an agitation dredging system, therefore, depended on two factors:

a. The ability of the device to raise material in the water column.

b. The flushing characteristics of Harbour Town Marina.

5. The agitation device tested was configured as shown in Figure A5. It consisted of four 2-in. pipes each 43 ft long radiating at 90 deg to each other from a central hub. Each pipe had approximately 129 holes drilled into it, each 1/16 in. in diameter and arranged in groups of three at 1-ft intervals along the pipe. The pipe array was laid on the bottom with the holes directed downward as shown in Figure A6. Air from a shore-based 30-hp compressor was fed via a flexible hose to the central hub, where it dispersed to the pipes and exited through the holes. The intended effects of the device, according to its designer, Dr. P. Bruun,* were to:

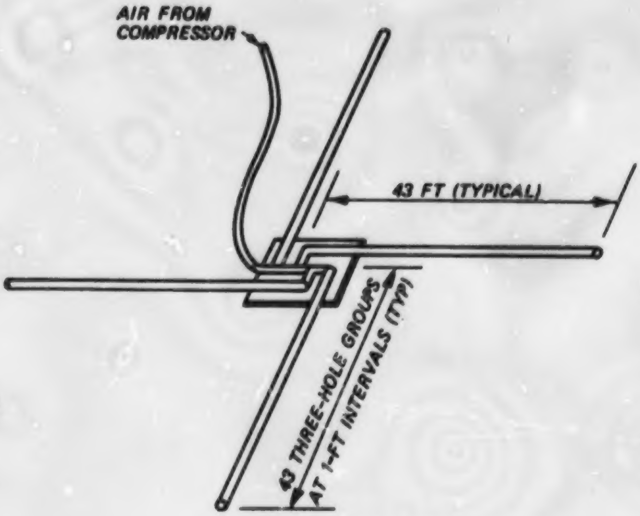

Figure A5. Agitation device, Harbour Town Marina

a. Prevent consolidation of settled material.

b. Erode material in the vicinity of the pipes through bottom currents induced by the air plume.

6. A limited water quality monitoring program was required for the tests by various regulating agencies. The test periods were restricted to 10 ebb tide flows over a maximum 2-month period. Sea Pines Plantation Co., which owns Harbour Town Marina, funded the construction and testing of the device as well as water quality monitoring. The US Army Engineer Waterways Experiment Station (WES) added a small amount of funding to expand the turbidity monitoring coverage. Water quality monitoring was conducted by Dr. H. L. Windom of Savannah, Georgia.

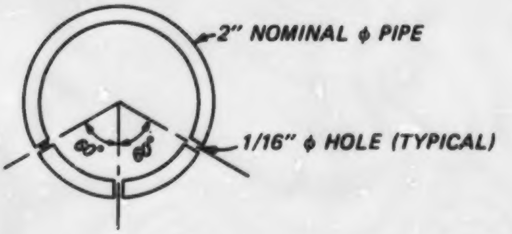

Figure A6. Layout of holes in pipe cross section, agitation device, Harbour Town Marina

7. Turbidity samples were taken at surface, middepth, and bottom for

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* P. Bruun. 1974 (May). "Air-Bubble Project," unpublished letter report.

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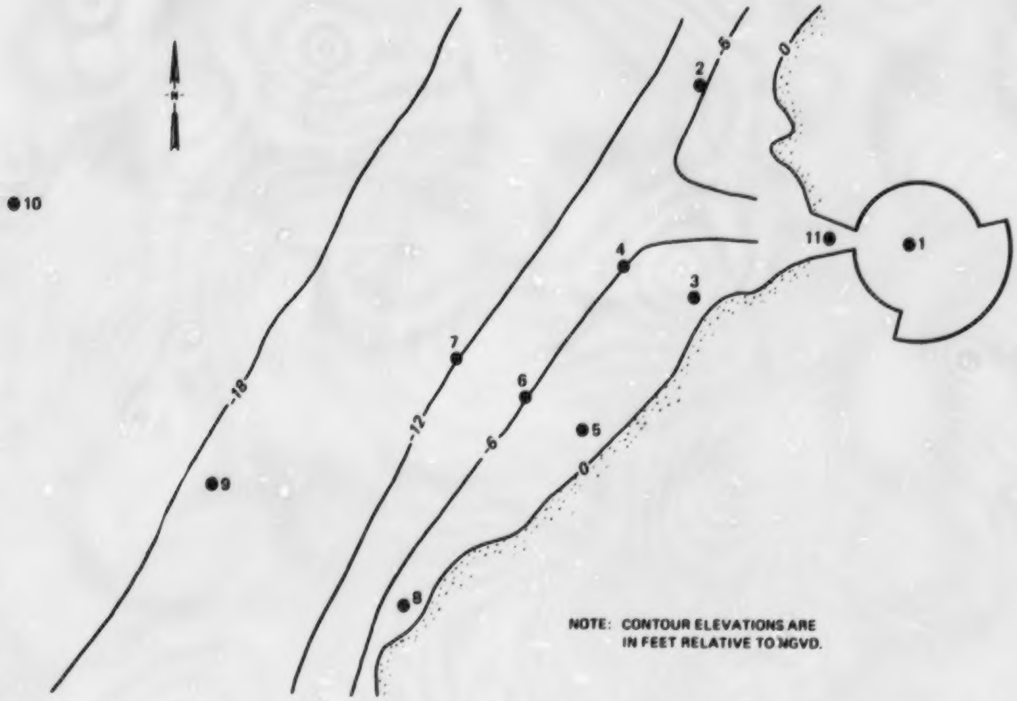
each of 11 stations located as shown in Figure A7. The stations were arranged to intercept the expected path of a turbidity plume exiting the harbor on ebb tide. Complete coverage of these stations was made 1 day before testing to check ambient conditions and on 3 days during testing. Results of these tests are summarized in Figure A8. All turbidity values are given in nephelometric turbidity units (NTU). In addition to this coverage, some detailed samples were taken at various times at sta 11 in the harbor entrance. Sampling across the entrance is summarized in Figure A9, and sampling versus time in the middle of the entrance is shown in Figure A10.



NOTE: CONTOUR ELEVATIONS ARE IN FEET RELATIVE TO NGVD.

Figure A7. Location of turbidity measuring stations, Harbour Town Marina

8. Most turbidity values in Figure A8 appear to be within the ambient range except for those at sta 2 and 4-8 on 17 March. The horizontal and vertical distribution of these values, however, indicates that this turbidity "plume" probably originated in Calibogue Sound. First, the turbidity extends through sta 2, which is upstream of Harbour Town Marina on an ebb tide. Second, the turbidity shows a gradual, steady increase from surface to bottom, characteristic of a well-mixed suspension. It is doubtful that such a degree of mixing would exist in a plume resuspended by the device tested. Third,

sta 1 and 11 in the harbor and entrance channel show no increased turbidity. Finally, a strong wind had been blowing from the northwest quadrant for several days before 17 March, possibly causing resuspension of material in shallow areas.

9. The only evidence in Figure A8 of some turbidity generation by the agitation device is on 15 March at sta 1, 11, and 3. The bottom samples of these stations and the middepth sample of sta 11 show turbidity values significantly higher than those obtained at other stations. The pattern suggests a "slug" of suspended material moving out of the harbor in the lower part of the water column. The cross-sectional sampling taken in the entrance channel at 0900 hr of the same day (Figure A9) does not show such an increase, however.

10. Figure A10 gives three plots of turbidity over depth versus time for sta 11. In the 11 and 17 March plots, no significant increase in turbidity is noted subsequent to starting the agitation device. In these and all other March tests, the device was operated in one location on the harbor bottom. In the April test, however, the device was pulled along the bottom of the harbor while being operated. The result was a dramatic increase

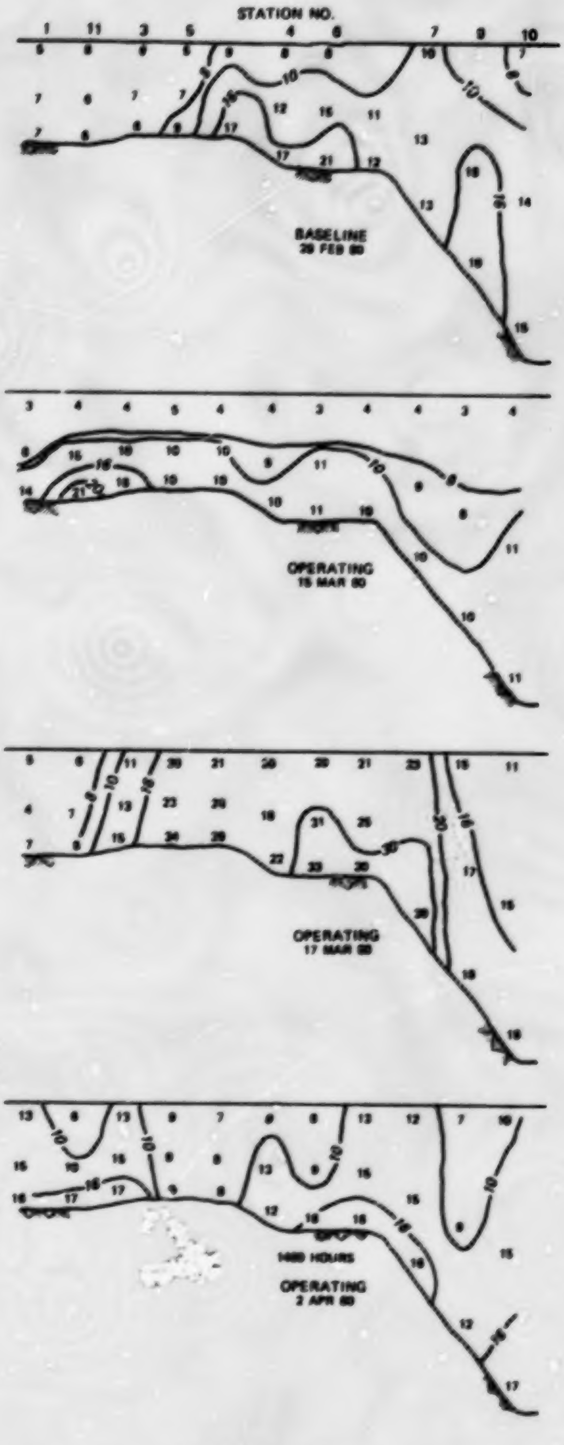
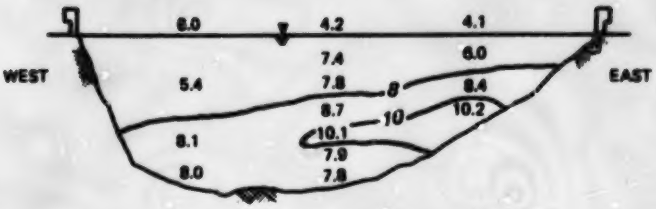


Figure A8. Turbidity (NTU) at sampling stations, Harbour Town Marina


6.0
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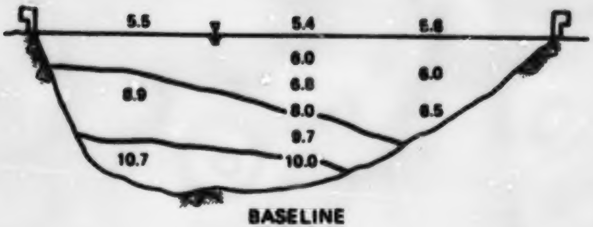
7.8

OPERATING
15 MAR 80


5.5
5.4
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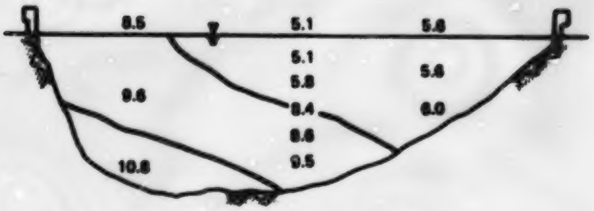
BASELINE
19 MAR 80
(1020 HOURS)


8.5
5.1
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5.1
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9.6
8.4
6.0
8.6
10.8
9.5

OPERATING
19 MAR 80
(1110 HOURS)

Figure A9. Turbidity (NTU) in entrance channel cross section, Harbor Town Marina

in turbidity at sta 11 beginning about 2 ft below the water surface. The increase lasted approximately 2 hr, and by 1400 hr, when data were taken for the Figure A8 plot, turbidity had subsided to near ambient. Although this mode of operation had obviously agitated bottom material in the harbor, evidence from previous tests seems to indicate that dragging, not air release, was the primary agitation mechanism.

11. A phenomenon was observed during these tests which although not directly connected to agitation dredging, is part of the local conditions that must be considered. On 14 March, winds were blowing from the west at 20 to 25 knots. A distinct turbidity plume was observed entering the harbor against

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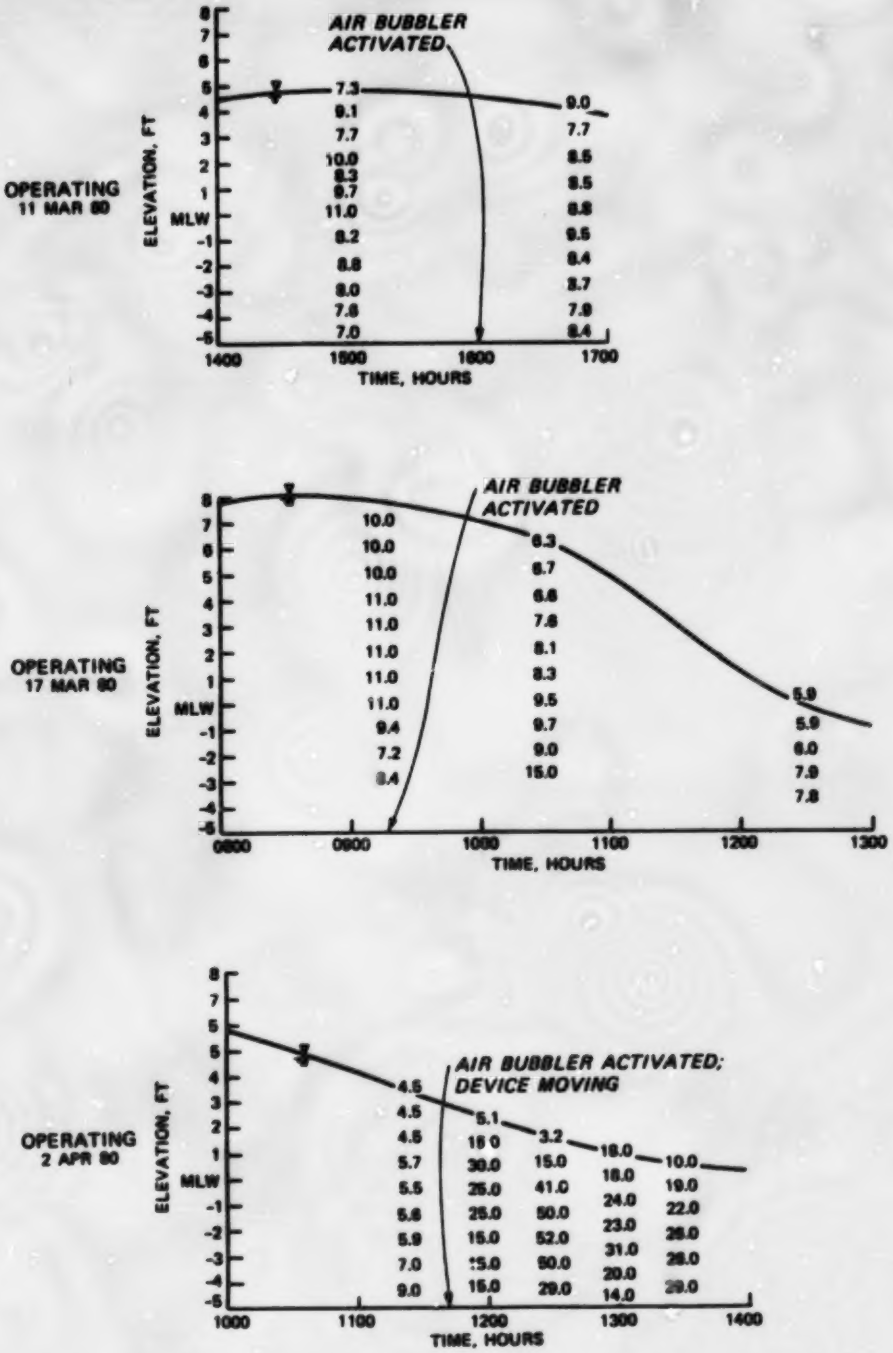


Figure A10. Turbidity (NTU) at sta 11, Harbour Town Marina

the ebb tide (Figure A11). Sampling in the plume tip revealed that turbidity was concentrated near the surface, with underlying waters having ambient values. A possible explanation for this phenomenon is that waves generated by the wind agitated material in nearby marshes and mud flats. The agitated material was then carried into the harbor entrance by mass transport, which rough calculations show would predominate over the weak ebb currents at the surface. Under such circumstances, turbidity movement into the harbor would be even more pronounced on flood tide. Depending upon the frequency of significant westerly winds, such a mechanism could be an important source of harbor shoaling.

## Delaware Estuary

12. The Delaware Estuary is located on the east coast of the United States, forming a boundary between New Jersey and Pennsylvania and Delaware (Figure A12). Navigation-related improvements have been made in the estuary since 1836, since it provides sea access to old and important ports such as Philadelphia and to industry in the Delaware Valley. Information in this description of the estuary and the history of agitation dredging there was gathered primarily from five sources: Wicker (1965, 1967) and the US Army Engineer District, Philadelphia (1967, 1969, 1973).

13. The estuary, which consists of Delaware Bay and the Delaware River, is a relatively simple, funnel-shaped system that is tidal up to Trenton, New Jersey (mile 130). Shortly upstream of Trenton, the riverbed rises at a steep gradient that precludes further tidal penetration. The salinity regime is well mixed under normal conditions, meaning there is no saline wedge to precipitate high rates of localized sedimentation. Salt water normally intrudes to approximately mile 75 but can reach Philadelphia during times of low freshwater discharge. When freshwater discharge is high, salinity may not be present above mile 65. Mean cross-sectional current velocities are fairly constant over the length of the estuary, in the range of 1.8 to 2.3 fps maximum. Maximum values within a cross section may reach 4 fps.

14. Sediments in the estuary above mile 57 can be characterized as mixtures of fine-grained particles with various amounts of fine sand included. In a shoal between miles 57 and 63, for instance, fine sand constitutes approximately 50 percent of the bottom sediment. Conversely, at Marcus Hook Shoal

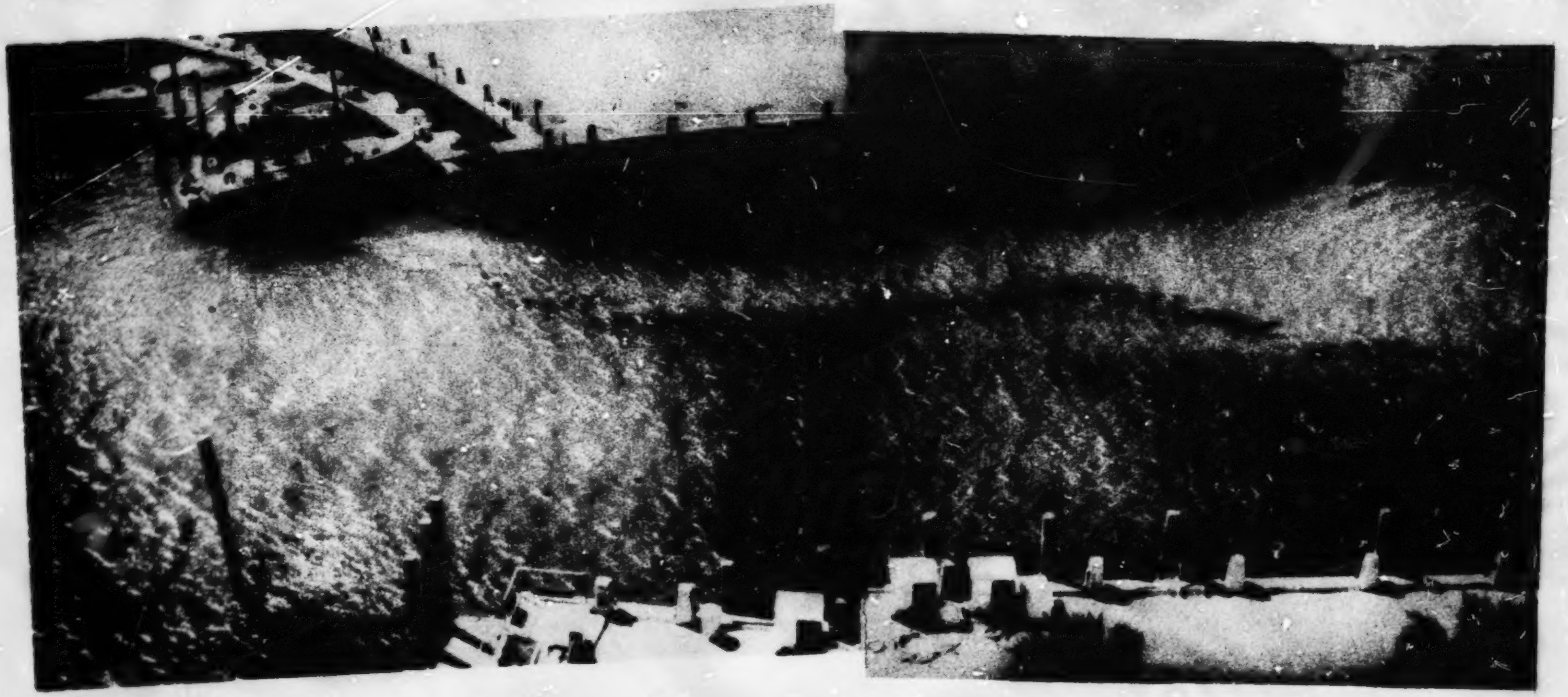


Figure A11. Surface turbidity plume entering Harbour Town Marina

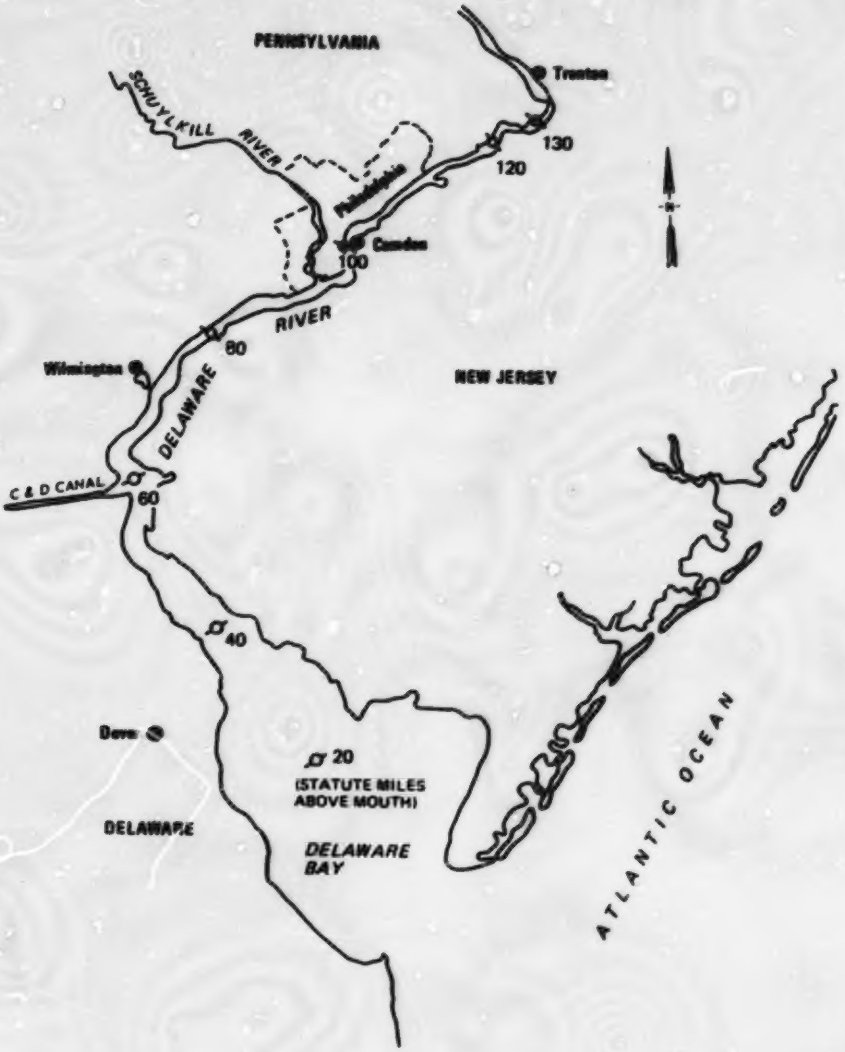

PENNSYLVANIA
Trenton
SCHUYLKILL RIVER
Philadelphia
130
120
N
Camden
100
RIVER
80
Wilmington
DELAWARE
NEW JERSEY
C & D CANAL
60
40
Dover
20
(STATUTE MILES ABOVE MOUTH)
DELAWARE
DELAWARE BAY
ATLANTIC OCEAN

Figure A12. Delaware Estuary

between miles 71 and 80, the sediment is almost totally fine-grained with a significant organic content. Clay minerals comprise a relatively small percentage of the mineral constituents in most locations except Marcus Hook Shoal. There, clay and quartz each provide about 25 percent of the total, and diatoms (silt-size algae with silica skeletons) form 18 percent. The Marcus Hook sediments are important to present-day dredging operations, since the Marcus Hook reach now accounts for 40 percent of the total navigation maintenance dredging. Wicker (1965) indicates that other areas, especially in the vicinity of mile 50, may have had greater shoaling importance in past years.

15. A 40-ft-deep navigation channel is maintained from the Atlantic Ocean to approximately mile 128, where it reduces to 35 ft. Six anchorage basins are located along the channel between miles 57 and 104. Virtually all of the estimated 8,200,000 cu yd per year of shoaling in the channel and anchorages takes place above mile 57. A primary reason for this, and one which directly affected the use of agitation dredging in the Delaware, is the existence of a tidal velocity null point in the vicinity of miles 40 to 50 (the exact location varies with freshwater discharge and the technique used to define the null point). Upstream of this null point, bottom currents in the ebb direction are predominant. Downstream, bottom currents predominate in the flood direction. In some estuaries, this predominance would continue to the estuary mouth. In the Delaware, however, a second null point is in the range of miles 15 to 20. Downstream of this second point, bottom ebb currents again predominate. The effect of these null points on shoaling in the Delaware is twofold:

- a. Net sediment movement from the Delaware River into Delaware Bay is prevented by the upstream null point.
- b. Net sediment movement from the Atlantic Ocean into Delaware Bay is also prevented or at least minimized by the ebb predominance downstream of the second null point.

Hence, shoaling in the bay part of the estuary is minimal since the major sediment sources are excluded. More importantly, the sediment load carried by the Delaware River is retained above the upper null point, resulting in a large dredging requirement and substantial disposal problems.

16. Hydrodynamics in the Delaware Estuary were not well understood when Corps of Engineers hopper dredges began working there in 1905. Between 1905 and 1915, hopper dredges operated in the Delaware in a primarily agitating mode. During ebb currents, the dredges would pump continuously. The hoppers would soon fill, and subsequent pumping produced a continuous sediment flow overboard at the water surface. At slack water, the dredge would dump its hopper into a free-dump area outside the navigation channel, and then dredge and dump without overflow during flood tide. The reasoning behind this early example of "hopper overflow" agitation dredging was that the agitated material would settle largely in areas of the estuary outside the channel and not be further disturbed due to the relatively weak currents there. The portion which resettled in the channel would, because it had been initially "disturbed," be moved more easily by stronger channel currents and eventually

carried into the ocean by the assumed predominant ebb flows. Even though material deposited in the free-dump areas was soon carried away by currents and undoubtedly redredged several times, it was reasoned that eventually these sediments would be carried out to sea and the estuary would become self-maintaining through hydraulic modifications wrought by dredging.

17. By 1915, it became obvious that agitation dredging and dumping in the estuary were not making any progress toward reduced maintenance requirements; in fact, the authorized depths could not be maintained. A new operational mode was then introduced, in which hopper dredges dumped material in rehandling basins where it was redredged and pumped to land disposal areas by cutterhead dredges. Emphasis was placed on efficiency of the operation, which led the hopper dredge masters to seek the greatest density they could achieve in each hopper load. They accomplished this by allowing overflow of the finer portion of dredged sediment and by lowering their drags to undercut the loosely consolidated upper bottom layers, dredging instead the denser material beneath. Both practices amounted to agitation dredging, the latter because the draghead generated a significant disturbance as it moved along beneath the sediment surface. In addition, studies of the rehandling basins showed that a large amount of material dumped there was carried away by currents before it could be rehandled. All told, less than 50 percent of the material handled by hopper dredges in the total dredging process was placed in land disposal areas.

18. Beginning in 1954, radical alterations in dredging methods were introduced for the Delaware Estuary. Procedures such as hopper overflow and draghead undercutting were minimized. Hopper loads, instead of being dumped back into the estuarine system, were pumped into a "sump rehandler," a vessel with a large hopper and pumps to move the load to land disposal sites. This procedure was further streamlined in 1963, when hopper dredges began pumping their loads directly to disposal areas, eliminating the sump rehandler.

19. The preceding history of modern dredging operations in the Delaware, though brief, shows how they have steadily evolved toward methods that: (a) minimize agitation during dredging and (b) remove as much of the dredged material as possible from the estuarine system. Dramatic improvements were made in maintenance results and requirements because of this evolution. Before 1954, controlling depths in the waterway either could not be maintained at the authorized values or else could be achieved only at enormous effort. It took 21 years (1910-1931) to achieve a depth of 35 ft. Efforts were begun

in 1940 to lower this to 40 ft, but the best that could be done by 1954 was slightly more than 36 ft. The more intense the dredging effort, the greater the shoaling rate became. However, within 9 years after beginning sump rehandling operations, annual maintenance dredging volumes had been cut by more than half and a 40-ft channel could be maintained. Although the cost per cubic yard in 1947-1949 dollars had increased by 53 percent, the total cost of dredging was only 67 percent of its former amount. The use of direct pumpout hopper dredges further reduced total costs and also lowered the cost per cubic yard.

20. The primary reason behind these improvements in dredging was an increase in bottom sediment density due to the elimination of agitation. When material was being agitated by hopper overflow, the hoppers retained much of the coarser sediment fraction and very little of the fines. Also, draghead agitation (or any type of bottom agitation) disturbed fine material more easily than coarse. Sediment transported out of the free-dump areas or rehandling basins by currents tended to have a higher percentage of fines than that which remained. When all this disturbed fine material resettled, it had an initial density as low as 1,050 g/ℓ. Thus a certain volume of sediment with a density of, say, 1,200 g/ℓ before disturbance could occupy a larger volume after resettling, even though part of the solids had been removed. The hydraulic dredging process is much less efficient when digging such light material, since a greater amount of unwanted water is being dredged and the material is more easily resuspended by the draghead.

21. Increased sediment density allowed hopper dredges to fill faster with a certain weight of material. Thus the weight rate of dredging by a given dredge was increased, allowing removal of material from the estuarine system at a rate equal to or exceeding the rate of new sediment influx. Prior to this, dredges had to cope with the resettled agitated material, which was dredged at a low weight rate, as well as new sediments introduced into the system. The basic premise behind agitation dredging, that the agitated material would eventually be "flushed" from the estuary, was shown to be invalid due to the upstream tidal velocity null point. The agitated material and sediment scoured from the free-dump areas and rehandling basins remained in the system, meaning that much of the dredging effort was a "closed loop," rehandling the same material over and over at low weight rates.

## Chinook Channel

22. Chinook Channel is located in the Columbia River estuary (Figure A13) on the north side approximately 7 miles from the estuary mouth. The channel, which is nominally 10 ft deep below mllw by 150 ft wide by 6,600 ft long, provides access to a mooring basin at Chinook, Washington. Tidal range in the Chinook Channel area is approximately 8 ft between mllw and mhhw, the maximum being 13 ft. Currents in the channel are predominantly tide-generated, with average and maximum ebb peak values of approximately 1.4 and 2.9 fps, respectively. Current patterns are influenced by flow in the estuary as a whole, so that current directions do not always follow the channel alignment. Figure A14, for instance, shows the directions of flow during maximum ebb current near Sand Island. Information on currents and other aspects of the operations described herein was obtained from the US Army Engineer District, Portland (1978).

23. Tests of an agitation dredging technique known as "propeller wash dredging" were conducted in Chinook Channel on 9 days in March and April 1976. In its simplest form, propeller wash dredging involves using the wake from a vessel's propellers to agitate bottom material. In this sense, most large vessels perform some propeller wash agitation in the course of navigating a ship channel. A more specific application employs an adjustable plate on the vessel's stern to deflect the propeller wash downward, thereby concentrating and directing the flow and producing some sediment transport by induced currents.

24. For the Chinook Channel tests, a 192-ft-long seagoing tug, the *Salvage Chief*, was used. The *Salvage Chief* was fitted with a 20- by 22-ft "S"-shaped door on the stern adjusted by six hydraulic cylinders (Figure A15). This door deflected the wash from two 55-in. propellers, each driven by an 1,800-hp diesel engine. The vessel draws 8 ft of water and is equipped with towing and anchor winches sufficient to service a six-point mooring system.

25. Agitation dredging tests using the *Salvage Chief* were conducted on 19 and 20 March and 19-26 April 1976. Figure A16 shows the test areas used for each period. Predredging, during dredging, and postdredging monitoring of various parameters was conducted to determine the physical and biological effects of the tests. An effort was also made to define the dredging cost.

26. The *Salvage Chief* operated for approximately 7-1/2 hr on 19 March, spanning an entire ebb flow, and for 2-1/2 hr on 20 March during ebb.

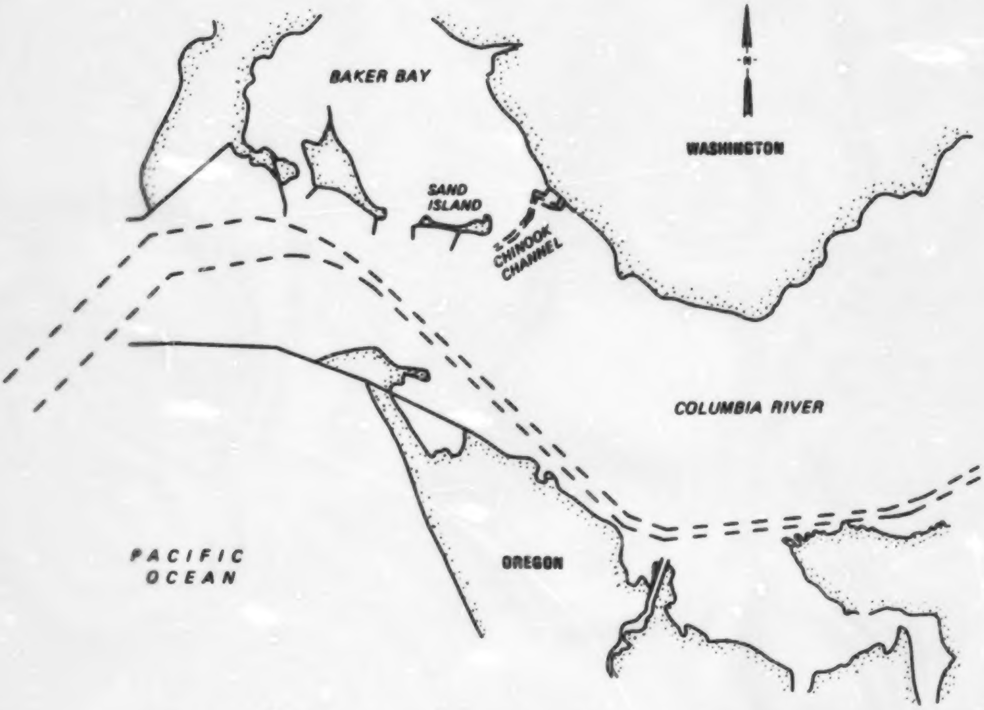

BAKER BAY
WASHINGTON
SAND ISLAND
CHINOOK CHANNEL
COLUMBIA RIVER
PACIFIC OCEAN
OREGON
N

Figure A13. Location map, Chinook Channel

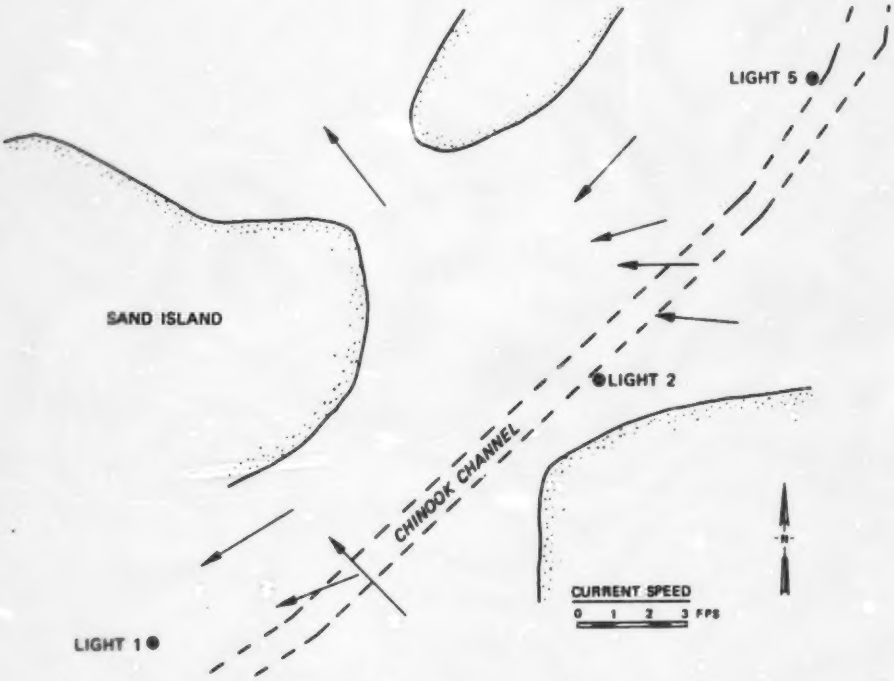

LIGHT 5
SAND ISLAND
LIGHT 2
CHINOOK CHANNEL
LIGHT 1
CURRENT SPEED
0 1 2 3 FPS
N

Figure A14. Maximum ebb current, Chinook Channel near Sand Island

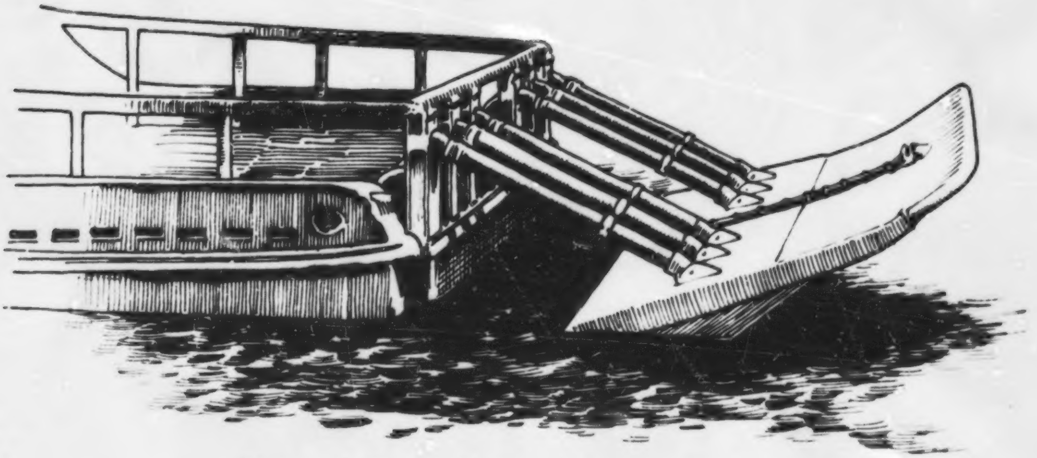


Figure A15. *Salvage Chief* deflector door

Operations on 20 March were abbreviated due to wind and wave action. These tests were considered trials to establish operating procedures. The vessel would cover a channel cross section in 90 min, spending 30 min each at both sides and the channel center line. After spanning a cross section, the *Salvage Chief* would move forward 100 ft on its anchor cables and repeat the process.

27. In April, the *Salvage Chief* worked in an area of the Chinook Channel near the Chinook Boat Basin. Actual dredging was accomplished for 64 hr during the 7-day period of this test, all during ebb tidal flows. Thirty hours of the 7 days were used in hydrographic surveying, moving the anchors and vessel, and responding to ship traffic. Seventy-four hours of the test period fell during flood tides, when agitation dredging was not attempted. One result of the April tests was modification of operating procedures during dredging. Instead of moving forward in 100-ft increments, the *Salvage Chief* was moved backward in 50-ft increments using the stern winch. Also, the channel cross section was swept by moving the vessel stern from side to side from a location on the channel center line, and total dredging time on each cross section was cut to 40 min. In all, five anchor lines on winches were needed to conduct this operation, as well as a full-time tender vessel to move anchors.

28. Results of these tests were reported in several ways. First, from hydrographic surveys conducted on 15 March and 28 April, it was concluded that a total of 44,500 cu yd of material was moved from the navigation channel by the *Salvage Chief* during the April tests, at a time-averaged rate of 470 cu yd/hr and a unit cost of $2.80 per cu yd. The hydrographic surveys also

revealed a major problem in regulating the scour patterns created by such operations. Holes 20 ft or deeper below mllw were created in some locations where preoperation depths were 6 to 11 ft. In other areas, the agitation produced a less concentrated removal of bottom material, although the bottom was noticeably more irregular after operations were concluded. In one 1,400-ft-long segment of the channel, only localized changes in the existing depths were recorded. However, in another 1,600-ft-long reach, the *Salvage Chief* was reasonably effective in attaining project depths of 10 ft or greater below mllw.

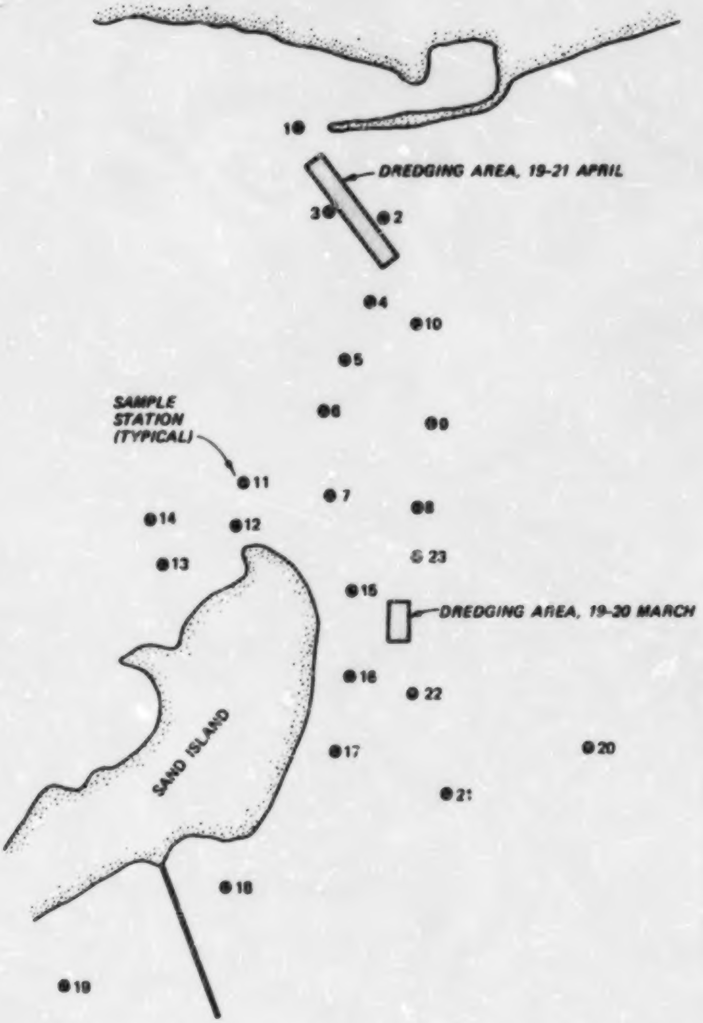


Figure A16. Sample stations, Chinook Channel

29. The second way of reporting test results was in terms of the operational characteristics of the *Salvage Chief*. Development of a procedure for "sweeping" the channel with the *Salvage Chief* was discussed in paragraph 27. In addition, the tests disclosed that the agitation/transport process was most efficient with approximately 8 ft of water under the *Salvage Chief's* keel. If water depths below the vessel decreased to 3 ft, a buildup of material occurred downstream of the propellers that inhibited the propwash flow pattern; the result was a noticeable loss in scouring rates. At the other end of the water depth spectrum, short tests conducted where depths below the vessel ranged from 22 to 42 ft revealed no conclusive bottom changes attributable to agitation. During one test, currents were measured

approximately 400 ft downstream of the *Salvage Chief* while it was operating with 10 ft of water below the keel. The total measured current was 3.0 to 3.5 fps. Subtracting an average tidal current for that time and location of 1.5 fps, it was concluded that the *Salvage Chief* was inducing a propwash current of 1.5 to 2.0 fps.

30. The final manner in which the *Salvage Chief* tests were reported was from the standpoint of environmental effects. Specifically, effects of the agitation dredging operations on epibenthic fish and invertebrates, water quality, turbidity, and benthic organisms were monitored and analyzed by personnel of the National Marine Fisheries Service of the National Oceanic and Atmospheric Administration. Trawl and sled tows were made along the entire length of the channel before the March and April tests, during the April tests, and 2 weeks after testing was completed. Benthic samples, water quality monitoring, and turbidity measurements were made at four stations located along the channel length prior to and during the April tests, then 6 and 14 months after completion of testing. This program of monitoring and sampling revealed an abundance of organisms such as flounder which are fairly high on the food chain, implying a similar abundance of lower level organisms. Large natural variations occurred in water quality, particularly salinity and temperature, as might be expected in the lower portion of a large estuary. Turbidity plumes of up to 2,580 ppm were measured during agitation dredging, at times when background levels ranged from 18 to 50 ppm. Such plumes caused temporary lowering of dissolved oxygen (D.O.) to marginal, but still acceptable, levels. Neither turbidity nor loss of D.O. was sustained for periods long enough to be considered harmful. The number of benthic organisms decreased in the agitation area during testing but had recovered somewhat by 6 months later. Benthic organisms downstream of the agitation area also decreased during testing but were at even higher levels after 6 months. The general conclusion was that agitation dredging had affected the benthic community on a short-term basis, but the degree of effect could not be determined. No direct evidence could be found of adverse effects on the epibenthic and demersal organisms measured by trawl and sled tows. Indices used to measure these organisms' diversity and richness were depressed somewhat during agitation operations, but had returned to preoperation levels or higher 2 weeks after testing. Overall water quality remained biologically acceptable during agitation dredging.

## Missouri River

31. A series of tests of agitation dredging in the Missouri River are described by Burke and Wyatt (1980). The tests involved an LCM (landing craft, mechanized) fitted with adjustable plates at the stern to direct propwash downward (Figure A17). This apparatus was tested at four locations in the Missouri River (Figures A18-A20) in five different operating modes. Elevation of the deflection plates was varied in three of the operating modes to determine the optimum position.

32. The Missouri River extends approximately 2,500 miles from the Rocky Mountains to St. Louis, Missouri (Figure A21). From St. Louis upstream to Sioux City, Iowa, a 9- by 300-ft navigation channel is authorized. The river is stabilized and regulated along this stretch into a series of bends averaging 2 miles in length, with no locks or dams interrupting flow. Currents in the main channel are fairly swift, ranging from 3.7 to 6 fps at normal discharges. Average bed material size is 0.25 mm (fine sand). Agitation

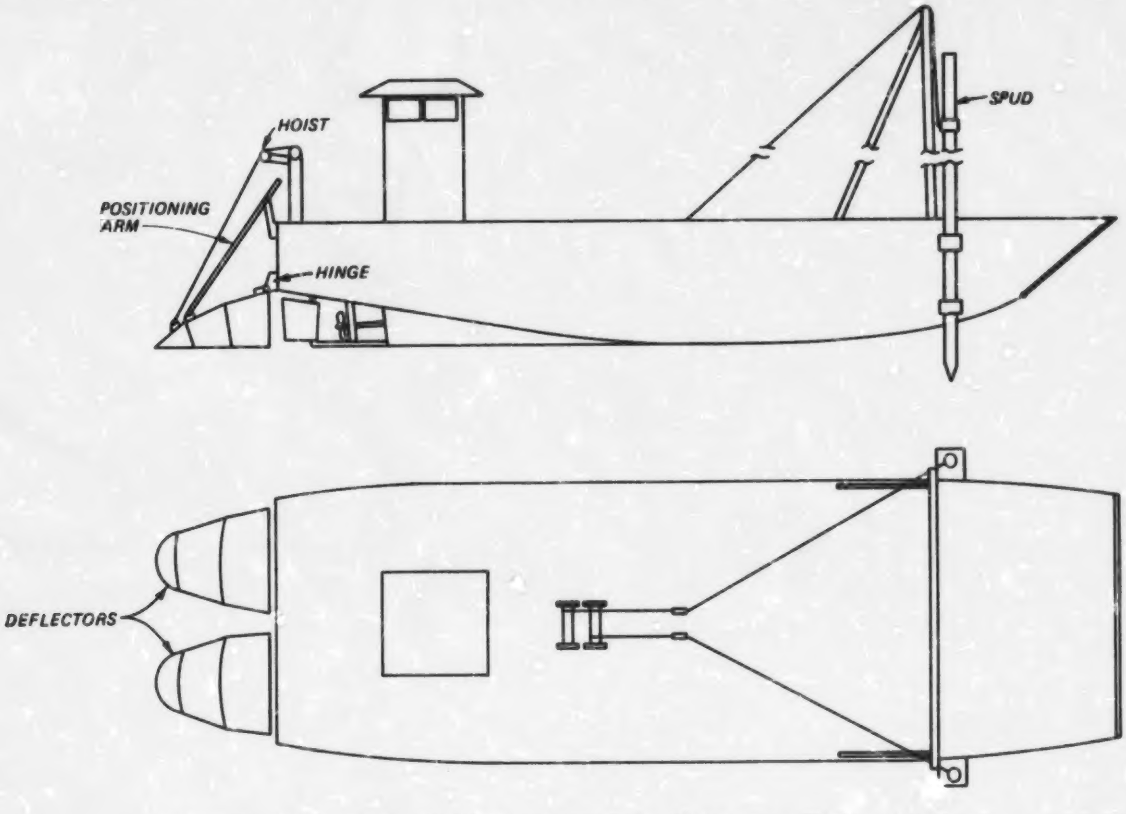


Figure A17. LCM agitation vessel, Missouri River tests

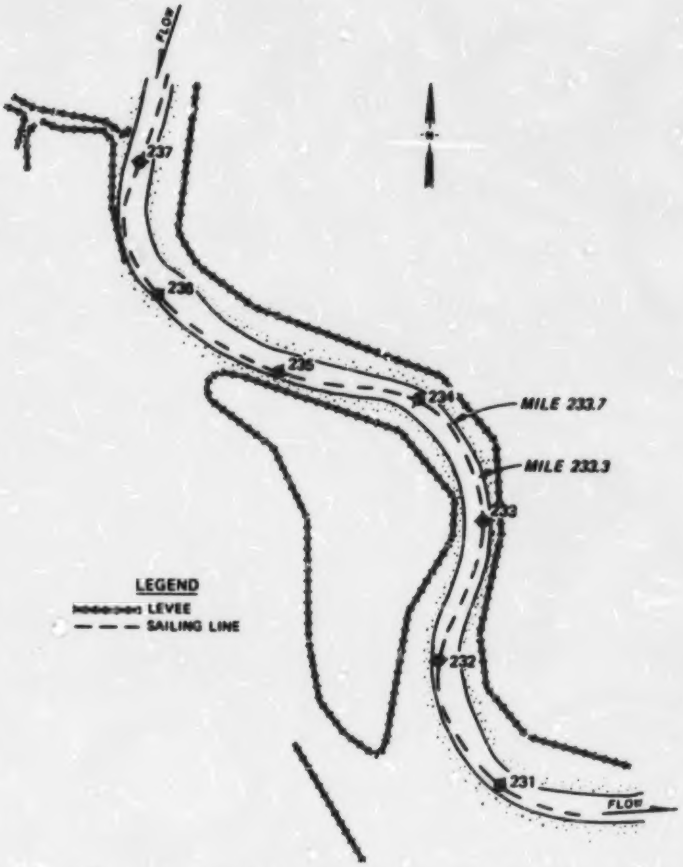

FLOW
237
236
235
234
MILE 233.7
MILE 233.3
233
232
231
FLOW
LEGEND
LEVEE
SAILING LINE

Figure A18. Agitation test sites, Missouri River, miles 233.3 and 233.7

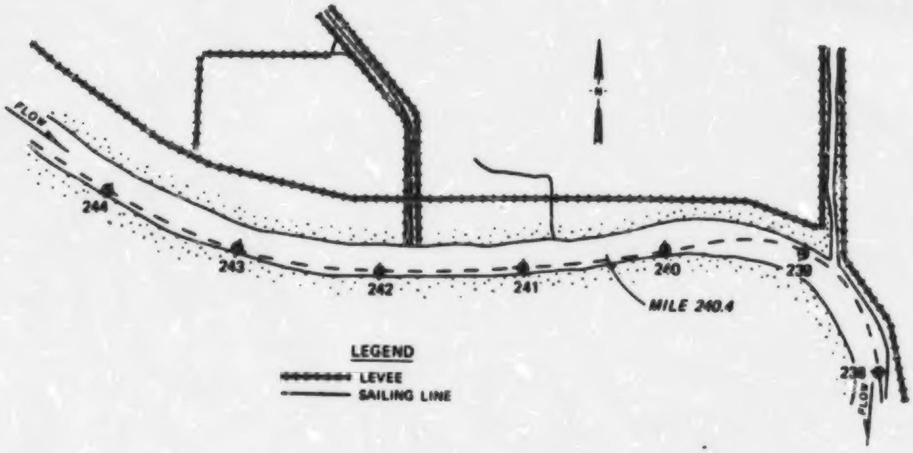

FLOW
244
243
242
241
240
239
MILE 240.4
238
FLOW
LEGEND
LEVEE
SAILING LINE

Figure A19. Agitation test site, Missouri River, mile 240.4

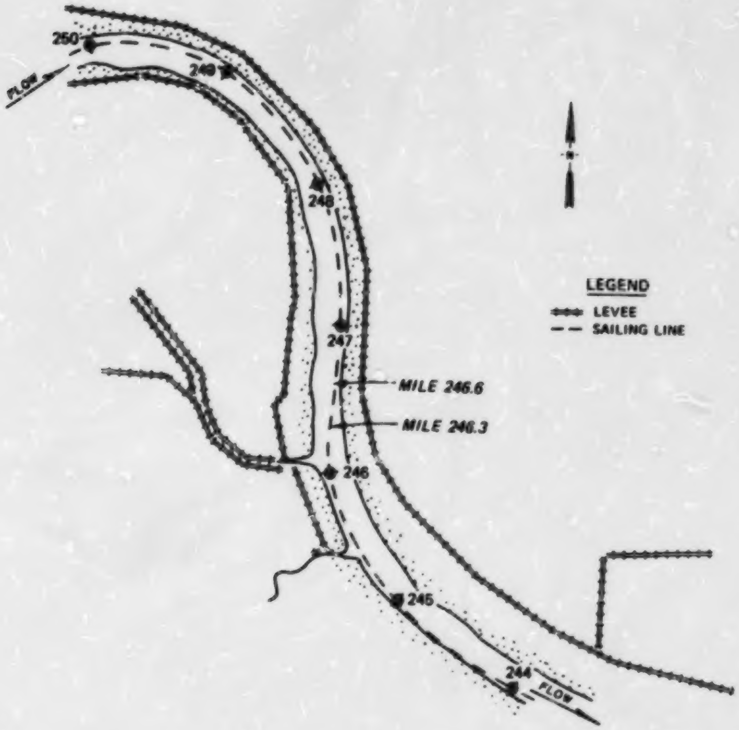

250
249
FLOW
N
248
LEGEND
LEVEE
SAILING LINE
247
MILE 246.6
MILE 246.3
246
245
244
FLOW

Figure A20. Agitation test sites, Missouri River, miles 246.3 to 246.6

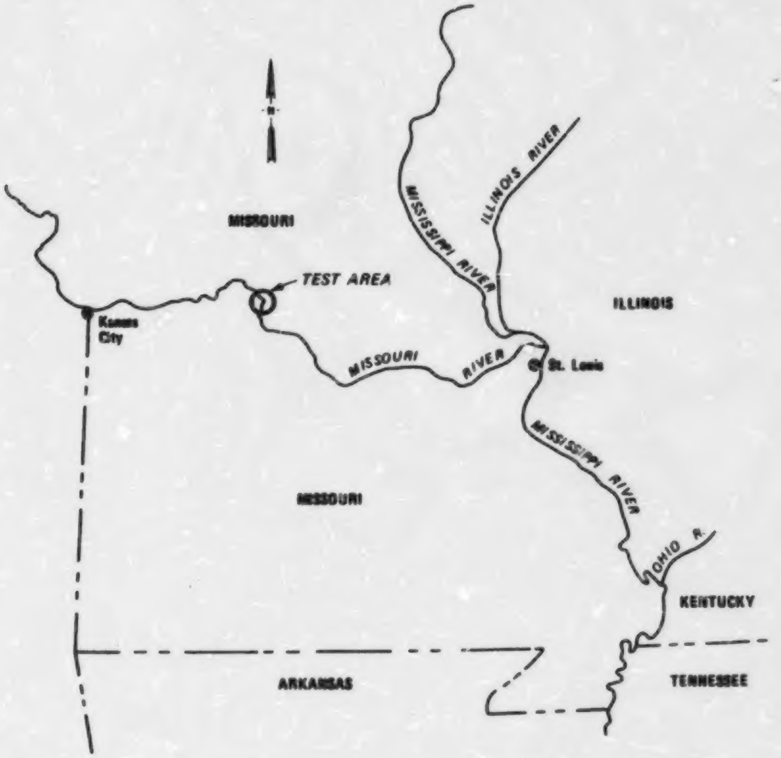

N
MISSISSIPPI RIVER
ILLINOIS RIVER
MISSOURI
TEST AREA
ILLINOIS
Kansas City
MISSOURI RIVER
St. Louis
MISSISSIPPI RIVER
MISSOURI
OHIO R.
KENTUCKY
ARKANSAS
TENNESSEE

Figure A21. Missouri River and agitation dredging test area

dredging tests were conducted in shoaling areas approximately 200 ft from the main channel, where velocities were considerably less.

33. Five modes of LCM operation were employed in the testing program:

a. LCM anchored on spuds; propwash directed downstream. Agitation began at upstream end of test area and proceeded downstream.

b. LCM anchored on spuds; propwash directed upstream. Agitation conducted at downstream end of test area.

c LCM run upstream through test area under own power; propwash directed downstream.

d. LCM pushed upstream through test area by towboat; propwash directed upstream.

e. LCM pushed downstream through test area by towboat; propwash directed downstream.

34. Table A1 summarizes the location, operating conditions, and scour depths for each mode of operation. Operational modes a and d seemed to give the best results in terms of depth increase, although the comparison of results is limited by obvious differences in test conditions. In operational mode a, the maximum increase in current velocity downstream of the LCM was measured with the deflector plates set at a 4-ft depth. This setting also gave the most rapid excavation rate. Currents measured 50 ft downstream of the LCM at this plate setting were approximately twice the ambient values. At 100 ft downstream, this increase had lessened, but bottom currents were still augmented substantially. Increases in suspended material were noticeable throughout the water column at 50 and 100 ft downstream. With the deflector plates set at 6 ft in operational mode a, a depth of scour was increased slightly, but material transport out of the area was reduced, resulting in a crescent-shaped berm around the downstream edge of the scour hole.

35. Operational modes b and c were relatively unsuccessful in removing bottom material, despite shallow-water depths and slightly higher currents in the test area than for operational mode a. In each mode, the effect of propwash was diminished by: (a) opposing river current for mode b or (b) relative movement of the LCM opposite to propwash direction for mode c. In operational mode d, although propwash was directed against a strong river current, the LCM was moved in the propwash direction. The towboat may have also contributed a significant amount of propwash in mode d. In mode e, the towboat/LCM combination was found difficult to control as it moved downstream through the test area.

## Table A1

## Agitation Locations, Conditions, and Scour Depths

| Mode of Operation | River Mile Location | Average Currents Test Area fps | Average Currents Main Channel fps | Average Predredge Water Depth ft | Deflector Plates Depth ft | Max Scour Depth ft | Comments |
|---|---|---|---|---|---|---|---|
| a | 246.6-246.3 | 2.8 | -- | 6.0 | 2 | 13 | 25-min agitation |
| | | | | | 4 | 14 | 10-min agitation |
| | | | | | 6 | 15 | 20-min agitation |
| | 233.7 | 2.7 | 5.2 | 8.5 | 2 | 10 | 45-min agitation |
| | | | | | 4 | NA | Mobility on spuds tested |
| | | | | | 6 | Negligible | 10-min agitation |
| b | 233.3 | 3.0 | 5.3 | 4.0 | 2 | Negligible | 45-min agitation |
| | | | | | 4 | NA | LCM moved upstream |
| | | | | | 6 | NA | LCM unstable |
| c | 233.3 | 3.0 | 5.3 | 4.0 | 2 | Negligible | 90-min operation |
| | | | | | 6 | Negligible | One pass through area |
| d | 240.4 | 5.6 | -- | 8.5 | 2 | ~1+ | Multiple passes through area |
| e | 240.4 | 5.6 | -- | 8.5 | 2 | ~0.5+ | Several passes through area |

## Pacific Northwest

36. Agitation dredging by propeller wash at nine different sites in the Pacific Northwest (Figure A22) in 1972 is described by the Portland District (1973). Operations at two of these sites, Port Orford, Oregon, and the Cowlitz River, Washington, are also discussed by Bechly (1975). At all locations, dredging was accomplished by an LCM called the *Sandwick*. This vessel is 74 ft long with a 21-ft beam and has two 34-in. propellers, each driven by two 165-hp diesel engines. The *Sandwick* was modified for propeller wash dredging by installing a 12-ft-long, 18-ft-wide deflector door on the stern, operated by three hydraulic cylinders. Design of the door was patterned after a patented system on the previously described vessel, the *Salvage Chief*. In addition to the door, two winches were installed to run through fairleads on each corner of the vessel, enabling four-point anchoring. The engine cooling system was improved and the deckhouse raised for better visibility. The vessel usually operates from a stationary position on four anchors, with engines running at three-quarter speed. When shoals are longer than 50 ft, several positions may be used to achieve the desired results.

37. At each of the nine projects shown in Figures A23-A31, the *Sandwick* was used to help remove small, isolated shoals that had formed in much larger navigation projects. Such shoals are exceptionally expensive to remove with conventional dredging equipment, due to high mobilization and demobilization costs relative to the amount of material dredged.

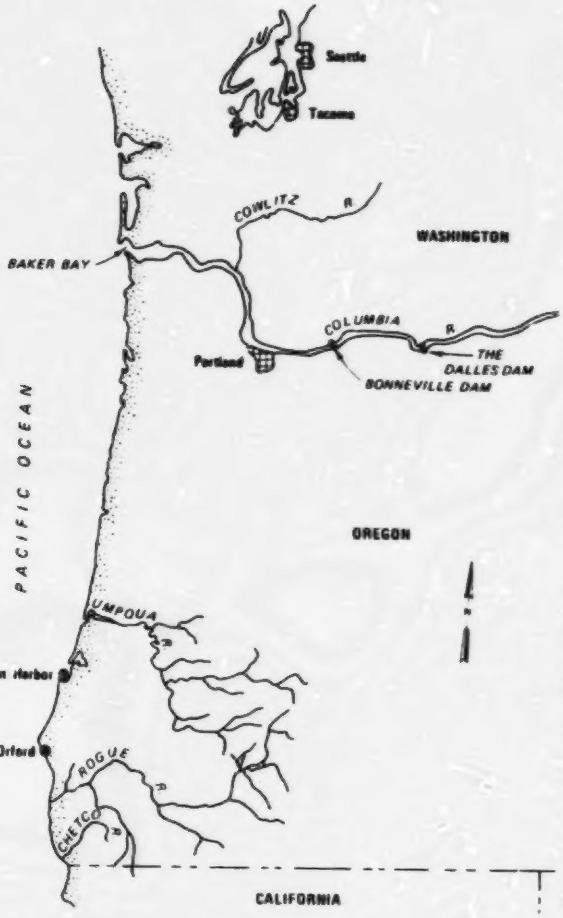

Seattle
Tacoma
COWLITZ R
WASHINGTON
BAKER BAY
COLUMBIA R
Portland
THE DALLES DAM
BONNEVILLE DAM
PACIFIC OCEAN
OREGON
UMPQUA
Charleston Harbor
Port Orford
ROGUE R
CHETCO
CALIFORNIA

Figure A22. Agitation dredging sites, Pacific Northwest

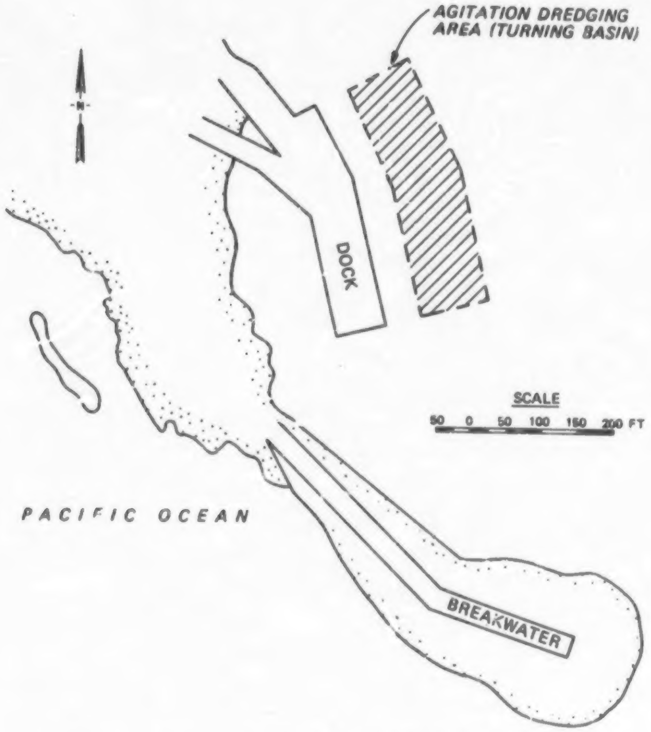

AGITATION DREDGING AREA (TURNING BASIN)
N
DOCK
SCALE
50 0 50 100 150 200 FT
PACIFIC OCEAN
BREAKWATER

Figure A23. Agitation dredging area, Port Orford, Oregon

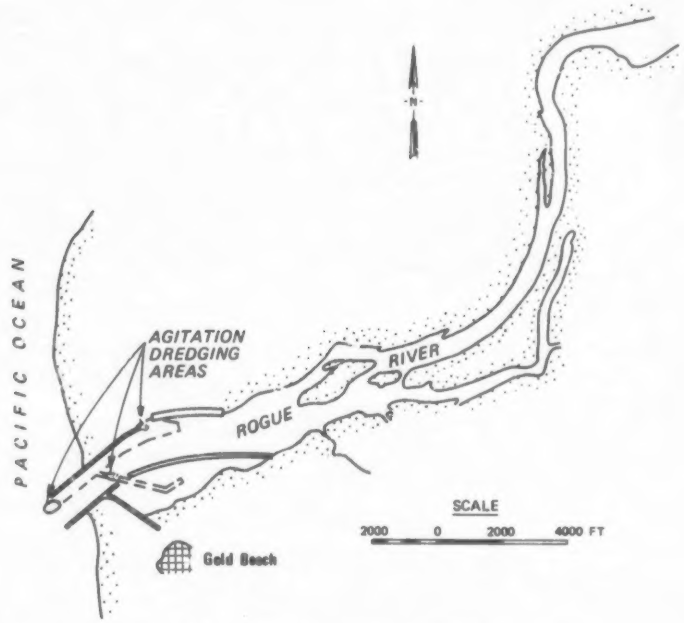

N
PACIFIC OCEAN
AGITATION DREDGING AREAS
ROGUE
RIVER
SCALE
2000 0 2000 4000 FT
Gold Beach

Figure A24. Agitation dredging areas, Rogue River, Oregon

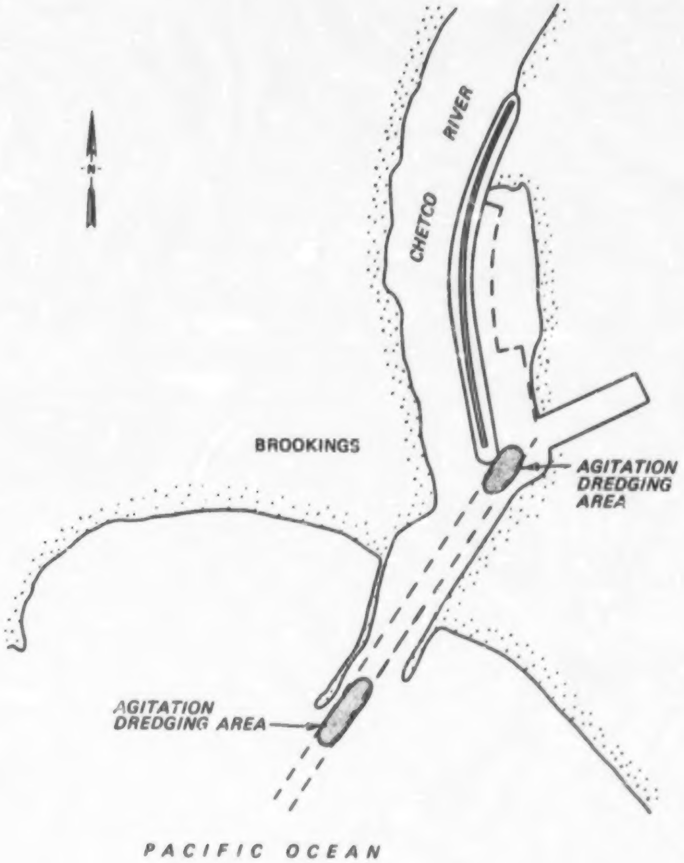

N
CHETCO
RIVER
BROOKINGS
AGITATION DREDGING AREA
AGITATION DREDGING AREA
PACIFIC OCEAN

Figure A25. Agitation dredging areas, Chetco River, Oregon

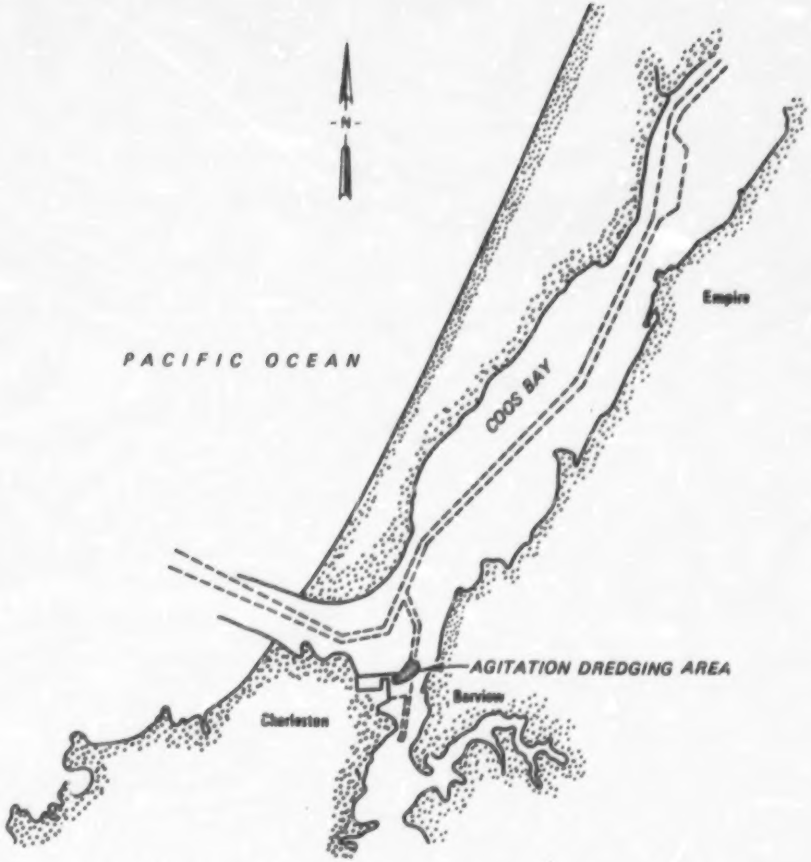

N
PACIFIC OCEAN
COOS BAY
Empire
AGITATION DREDGING AREA
Barview
Charleston

Figure A26. Agitation dredging area, Charleston Harbor, Oregon

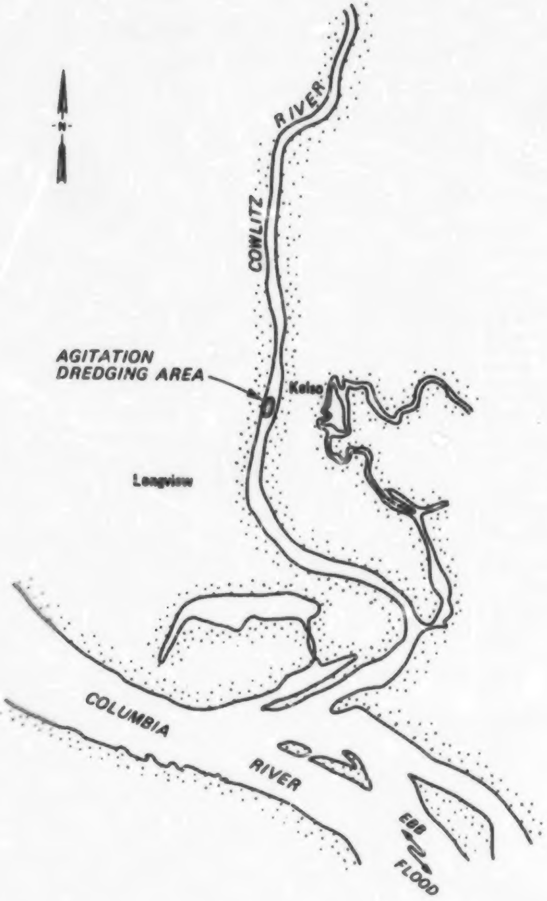

N
COWLITZ
RIVER
AGITATION DREDGING AREA
Kelso
Longview
COLUMBIA
RIVER
EBB
FLOOD

Figure A27. Agitation dredging area, Cowlitz River, Washington

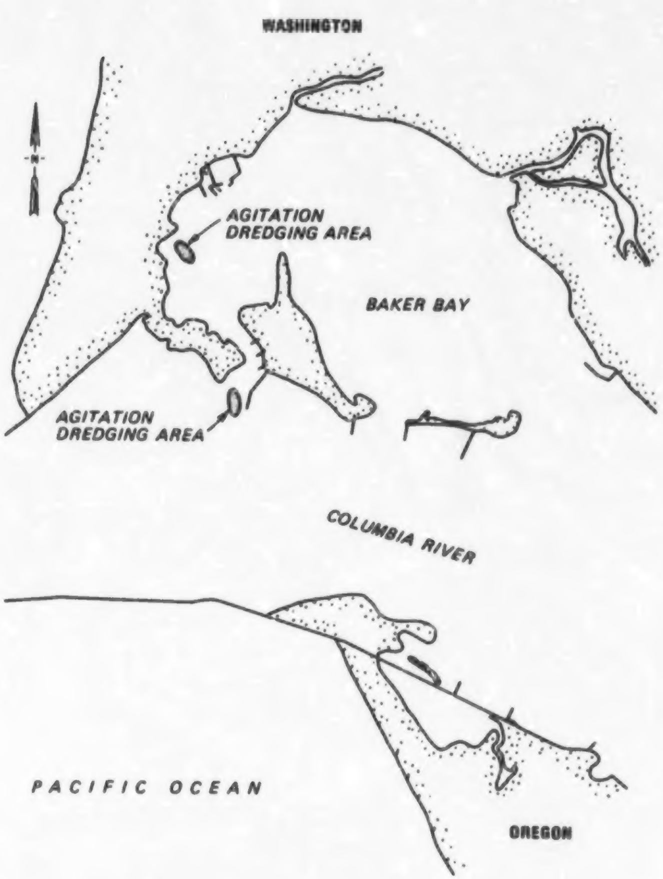

WASHINGTON
N
AGITATION DREDGING AREA
BAKER BAY
AGITATION DREDGING AREA
COLUMBIA RIVER
PACIFIC OCEAN
OREGON

Figure A28. Agitation dredging areas, Baker Bay

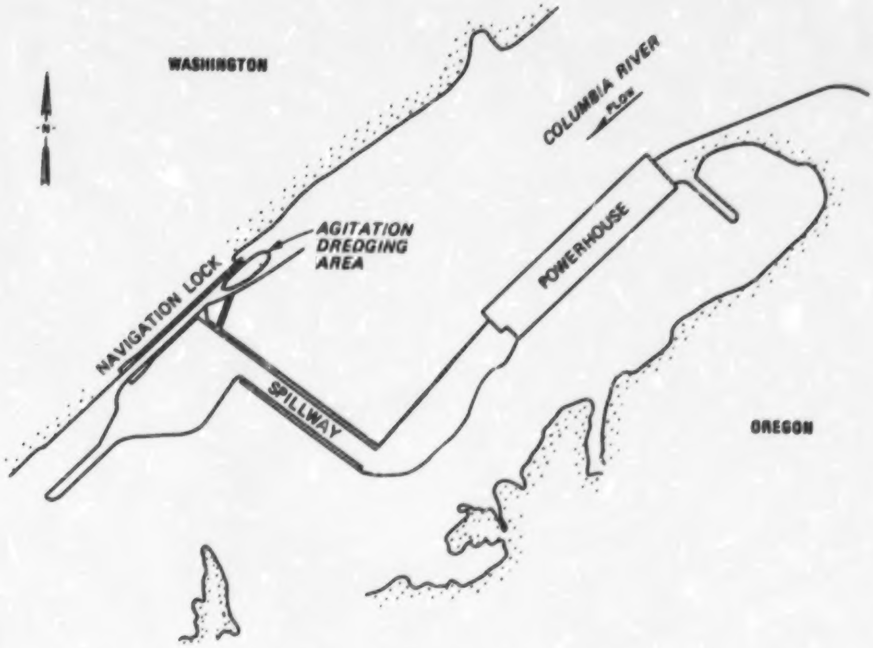

WASHINGTON
N
COLUMBIA RIVER
FLOW
AGITATION DREDGING AREA
NAVIGATION LOCK
POWERHOUSE
SPILLWAY
OREGON

Figure A29. Agitation dredging area, The Dalles Dam

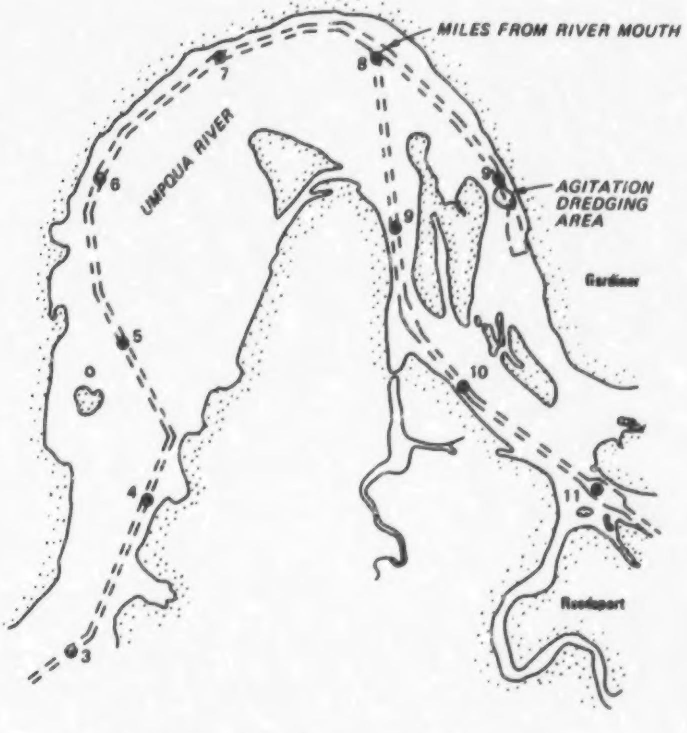

MILES FROM RIVER MOUTH
7
8
UMPQUA RIVER
6
9
9
AGITATION DREDGING AREA
Gardiner
5
10
4
11
Reedsport
3

Figure A30. Agitation dredging area,
Umpqua River

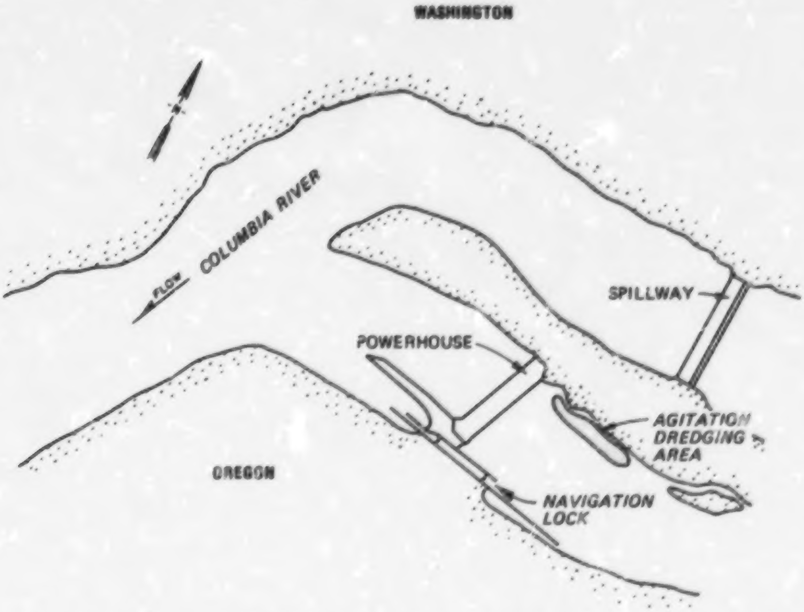


Figure A31. Agitation dredging area, Bonneville Dam

38. The projects provided a wide range of operating conditions and results, summarized in Table A2. Pre- and postoperational hydrographic surveys were made at several of the projects, allowing determination of the volumes dredged. The same projects were monitored for the amount of time actually spent in agitation dredging, so that average dredging rates could be computed. The following general conclusions can be drawn regarding the data in Table A2:

a. Average dredging rates were considerably higher in silt and fine sand than in coarser material.

b. Average dredging rates and depths of scour (difference between pre- and postdredge water depths) were generally greater in shallow than in deeper water for a given bottom material type.

c. Operating costs of the *Sandwick* compared very favorably with the costs of conventional dredging methods used previously on five of the nine projects.

39. In regard to conclusion c, it should be noted that *Sandwick* costs shown in Table A2 are based on an estimated "rental rate" for the vessel and crew. Experience gained during operations at the projects summarized in Table A2 indicated that a substantial increase in the rental rate was needed to cover higher maintenance costs. Despite this increase, the *Sandwick* would probably remain cost-competitive on these projects.

40. Operations at two of the sites (Rogue River and Chetco River)

68

## Table A2

## *Sandwick* Operating Conditions and Agitation Dredging Results

| Project Location | Bottom Material | Current fps | Average Water Depth | | Volume Dredged cu yd | Operating Time hr | Average Dredging Rate cu yd/hr | Cost/cu yd | | Comments |
|---|---|---|---|---|---|---|---|---|---|---|
| | | | Predredge ft | Postdredge ft | | | | *Sandwick* | Previous Methods | |
| Port Orford, Oreg. | Sand $d_{50} = 0.2$ mm | Nil | 9.0 | 13.1 | 12,000 | 47 | 255 | $0.63 | $6.00 | |
| Rogue River, Oreg. | Sand and gravel | Unknown | 21.3 inner shoal<br>10.5 bar | Unknown | Nil | -- | -- | -- | -- | Terminated due to anchoring problems on bar and need for repairs |
| Chetco River, Oreg. | Sand, coarse w/some fine | Unknown | 10.9 inner shoal<br>12.2 bar | Unknown | Unknown | -- | -- | -- | -- | Successful on inner shoal; anchoring problems on bar |
| Charleston Harbor, Oreg. | Sand, coarse | Unknown | 6.0 | Unknown | Unknown | -- | -- | -- | -- | Successful removal of sand spit encroaching on channel |
| Cowlitz River, Wash. | Sand and gravel $d_{50} = 0.5$ mm | 3-5 | 4.9 | 7.8 | 9,000 | 34 | 265 | 0.51 | 0.88 | |
| Baker Bay, Wash. | Coarse sand | Nil | 7.3<br>10.4 | 9.1<br>11.1 | 8,300 | 43 | 190 | 0.56 | 2.00 | Sand partially cemented |
| The Dalles Dam | Silt and fine sand | Nil | 13.6 | 16.3 | 18,000 | 28 | 640 | 0.24 | 1.30 | |
| Umpqua River, Oreg. | Silt and fine sand | 2-4 | 19.8 | 20.9 | 8,300 | 20 | 415 | 0.25 | 0.47 | |
| Bonneville Dam | Silt and fine sand | Unknown | 10.0 | Unknown | Unknown | -- | -- | -- | -- | Successfully moved shoal into deep water |

revealed problems in using the *Sandwick's* anchoring system in an area exposed to wave action. At the Rogue River mouth, anchors could not be used when seas were greater than 2 ft. Difficulties were encountered at the Chetco River with anchor cables breaking as the *Sandwick* moved under wave action. Another problem occasionally encountered was compacted or slightly cemented bottom material which could not be easily dislodged by propeller wash alone. Proposed methods for alleviating these problems and improving other aspects of the *Sandwick's* operations are discussed in Appendix A of the Portland District (1973) report.

41. Several aspects of the *Sandwick's* operations at the nine project sites deserve mention. At most sites, the vessel was used as the primary dredging vehicle, i.e., no additional dredging of the agitated material by other equipment was planned. This meant that the *Sandwick* often had to "chase" material to an acceptable location by repeated reagitation. In coarse material where existing currents were nil, sediment was displaced less than 100 ft by the *Sandwick* operating in one position. In fine material such as silt, the propeller wash currents might carry it 400 to 600 ft. At Charleston Harbor, the *Sandwick* was used to augment operations of the hopper dredge *Pacific* by displacing material into deeper water where the *Pacific* could dredge it. At the Bonneville Dam project, the *Sandwick* moved material from a shoal at the exit of a fish ladder to deeper water where it would be carried through the dam by existing currents. At The Dalles Dam site, the area cleaned by the *Sandwick* was the upstream approach to a lock. Shoal material was "chased" out of the lock approach into the forebay area of the main dam by starting at the lock gate and working upstream.

42. No formal environmental monitoring was conducted at the nine project sites. At each location, however, operations were coordinated with State and Federal resource agencies, and the *Sandwick* was observed in operation at several sites by representatives of such agencies. No broad objections were voiced to the propeller wash agitation method, although at several locations additional coordination was requested for continued or repeated work. Particular concerns were possible smothering of stationary mollusks such as clams and interference with spawning runs of fish such as steelhead trout or salmon.

## Tillamook Bay

43. Tillamook Bay is an estuary 3 miles wide by 6 miles long on the

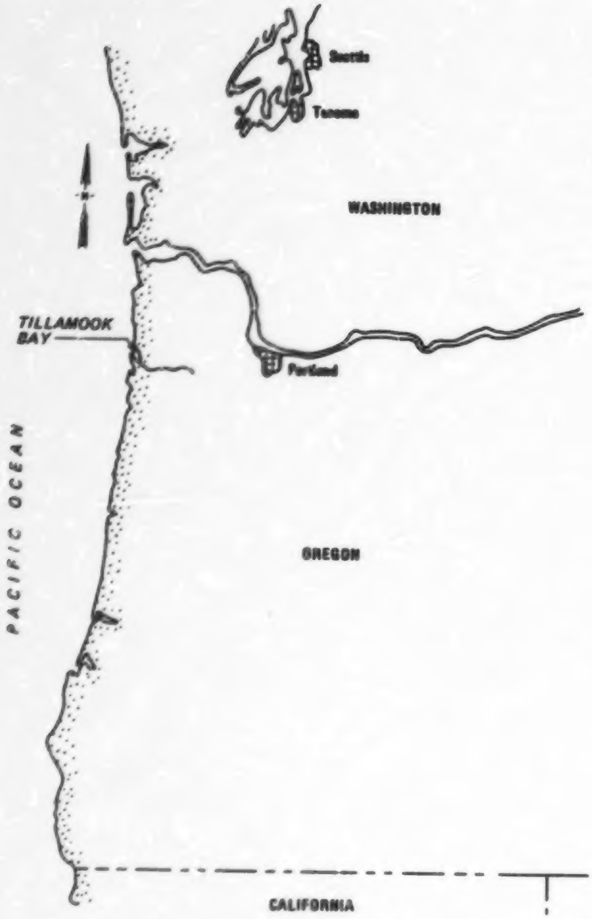


Figure A32. Tillamook Bay

coast of Oregon (Figure A32). The bay contains a number of oyster beds and fish spawning areas, and has experienced continued sedimentation problems for many years. Maintenance dredging has been required annually since 1959, primarily on the entrance bar and inner channel. Occasional dredging is needed in the Garibaldi boat basin and the Trask and Wilson Rivers.

44. Agitation dredging operations were conducted in Tillamook Bay in December 1973 using the vessel *Sandwick* described earlier. Concurrent with these operations, tests were conducted to determine their interaction with bay hydraulics, sediment chemistry and physics, water quality, and biological organisms. Results of these tests are given by Slotta et al. (1974). Measurements were also made of scour hole dimensions and velocity distributions in the vessel's wake. These measurements are reported in Slotta and Higgins (1976).

45. The *Sandwick* was operated in the navigation channel near the Garibaldi boat basin from 7-19 December 1973, in the hatched areas shown in Figure A33. This period was selected as one in which agitation dredging would have minimal effect on fish and other organisms. This period also coincided with high freshwater discharges into the bay, carrying large amounts of sediment. *Sandwick* operations were restricted to daylight hours during ebb flows, meaning that total dredging time was quite limited; in fact, less than 17 hr during the operating period were spent in actual dredging. In addition to time limits on agitation, inclement weather and operational problems combined

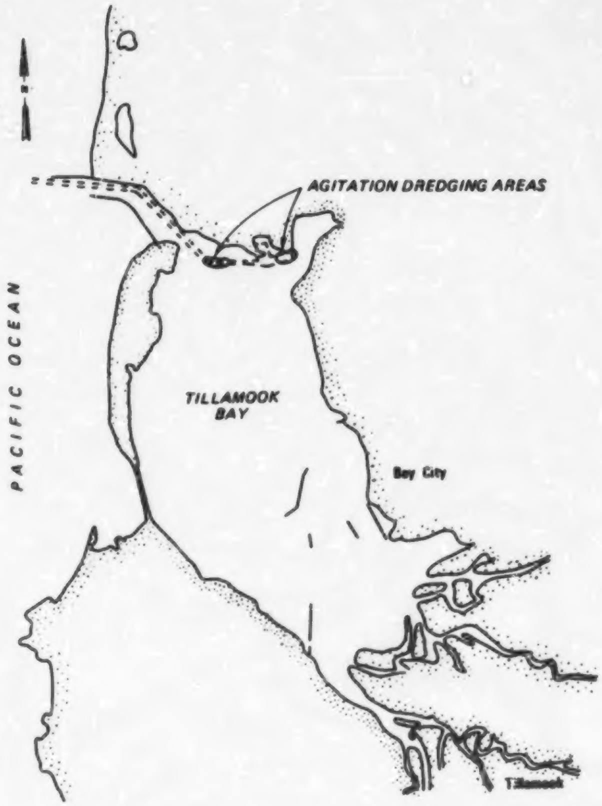
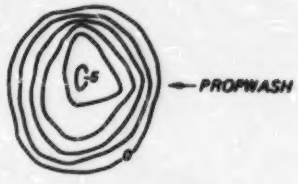


Figure A33. Agitation dredging areas, Tillamook Bay

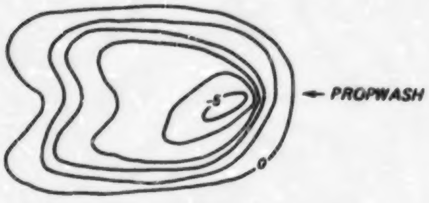
to decrease available dredging time. As a result of this short dredging period, only 5,900 cu yd of material was estimated to have been removed from the channel during the tests. However, this volume translates to an average removal rate of 350 cu yd/hr, which compares favorably with rates for similar circumstances described previously for the *Sandwick* in this appendix. Sediments in the dredging area were primarily sand with a median grain size of 0.25 to 0.35 mm. Predredging water depths ranged from +2 to -24 ft relative to mllw.

46. Because of the short operational period and small volume of material removed, it was difficult to separate changes caused by *Sandwick* operations from those due to natural processes. Probably the most successful effort in this sense was the measurement of scour hole dimensions and velocity profiles in the vessel's wake. Scour holes were created in tidal flats at high water and measured at low tide when they were exposed. Current measurements were attempted over the tidal flats in Tillamook Bay, but were frustrated by scoured debris damaging the current meters. Measurements made later in deeper water at Coos Bay, Oregon, form the basis for velocity profiles described herein.

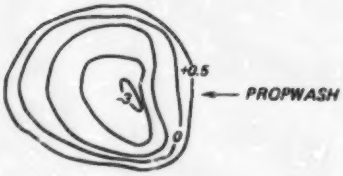
47. Shapes of three scour holes formed by the *Sandwick* are shown in Figure A34. These holes were made in sediment consisting of fine sand with a median grain size of 0.20 mm, in predredge water depths of 6.5 ft. For all three holes, the *Sandwick's* deflector door was lowered to its maximum angle with the horizontal of 20 deg, and engines were run at 1,500 rpm. Scouring was accomplished for the following times for each hole: (a) north hole, 150 sec, (b) central hole, 300 sec, and (c) south hole, 120 sec. The holes are



NORTH SCOUR HOLE



CENTRAL SCOUR HOLE



SOUTH SCOUR HOLE

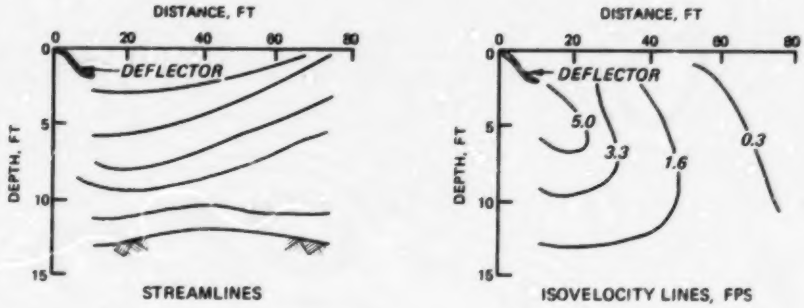
Figure A34. Scour hole shapes, Tillamook Bay; contour intervals are in feet, wtih zero contour representing prescour bottom

somewhat different in shape, with the central hole reflecting the effects of the *Sandwick's* twin propellers in its two-lobed downstream shape. It can be postulated that the rounder configuration in the other holes is due to their shorter scour times. An unexpected aspect of the scour holes' shapes was the absence of a significant berm of excavated material at the lip, as occurred in the Missouri River propwash tests described earlier. Maximum scour depth in the Tillamook Bay was 5 ft below existing bottom for the north and central holes and 3.5 ft for the south hole.

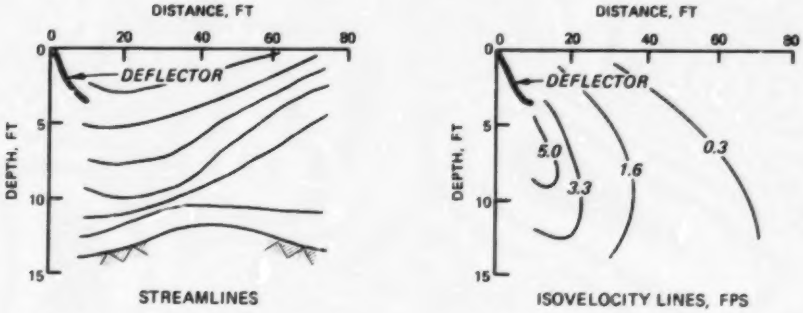
48. Figure A35 shows vertical streamlines and velocity profiles immediately behind the *Sandwick* for two angle settings of the deflector, at constant engine speed of 1,500 rpm. The maximum angle setting of 20 deg appears to produce higher velocities and more concentrated streamlines near the bottom than the 10-deg setting. Velocities 50 ft behind the *Sandwick* are an order of magnitude less than those reported as being caused by propeller wash in the Missouri River tests. However, water depths at the *Sandwick* test site were more than twice those in the Missouri River. In addition, Slotta (1976) noted the observed complexity of flow in the *Sandwick's* turbulent wake. This complex flow, which included vortices 6 to 7 ft in diameter, meant that any reported velocity profile was only an average of very unsteady short-term velocities.

49. An interesting result from the Tillamook Bay scour hole tests is shown in Figure A36, which plots the total volume of each hole versus the total scour time in seconds. Although there are only four data points, there is an obvious tendency for the excavation rate (slope of the fitted curve) to decrease rapidly with time. The fitted curve in Figure A36 is a second degree polynomial; at 50 sec its slope is 3,880 cu yd/hr, while at 400 sec the slope is only 253 cu yd/hr.

50. No significant environmental effects were measured during the



DEFLECTOR LOWERED 10 DEGREES



DEFLECTOR LOWERED 20 DEG

Figure A35. Streamlines and isovelocity lines behind *Sandwick*

Tillamook Bay tests that could be related to *Sandwick* operations. Resuspension of sediment by the *Sandwick* caused no measurable increase of soluble organic carbon, total sulfides, or ammonia in the water column, all of which are indicators of potentially adverse water quality changes related to sediment chemistry. A small increase in water column turbidity was noted, but could not be tied to the agitation dredging.

51. Results of biological monitoring before and after *Sandwick* operations were mixed. Mean total abundance of species <1.0 mm in size on and in the bottom changed, but the changes did not occur in patterns related to dredging. Abundance of organisms living above the bottom declined, but this was attributed mainly to high freshwater inflows and inclement weather conditions during post-dredging surveys. The latter factor caused an apparent decrease in sampling efficiency. Organisms

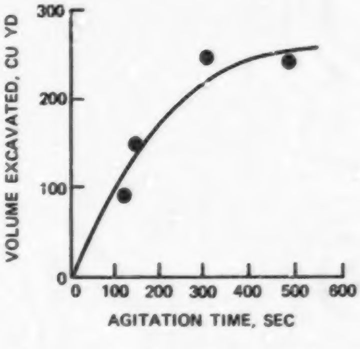


Figure A36. Excavation volume versus agitation time, Tillamook Bay

kept in cages near the dredging site showed no signs of acute toxicity or other changes resulting from *Sandwick* operations. No measurable increase of sedimentation on nearby clam beds was noted. The summary conclusion of biological monitoring was that natural environmental stresses during dredging outweighed any changes caused by *Sandwick* operations.

52. Concurrent with environmental effects monitoring, measurements were also made of tides, currents, circulation patterns, and changes in bottom sediment characteristics. Ebb currents in the vicinity of Garibaldi boat basin begin concurrently with a falling tide and persist for up to 1 hr after low tide. Current measurements were made in the period 12-14 April 1974. The maximum ebb current recorded during this time was 3.5 fps. Drogue and dye studies showed that the *Sandwick* was operating in a region of complex current patterns (Figure A37). In the area where the *Sandwick* dredged, ebb circulation appears split between a clockwise eddy to the east and currents flowing toward the bay mouth to the west. Also, an ebb current shear line was noted in these studies separating eddy-induced circulation to the east from higher velocity ebb currents to the west. Bottom drifters released in the dredging area and at other points in Tillamook Bay were recovered 1 to 10 days later either in the entrance channel or on adjacent ocean beaches. Therefore there may be a net oceanward bottom transport in Tillamook Bay as opposed to the

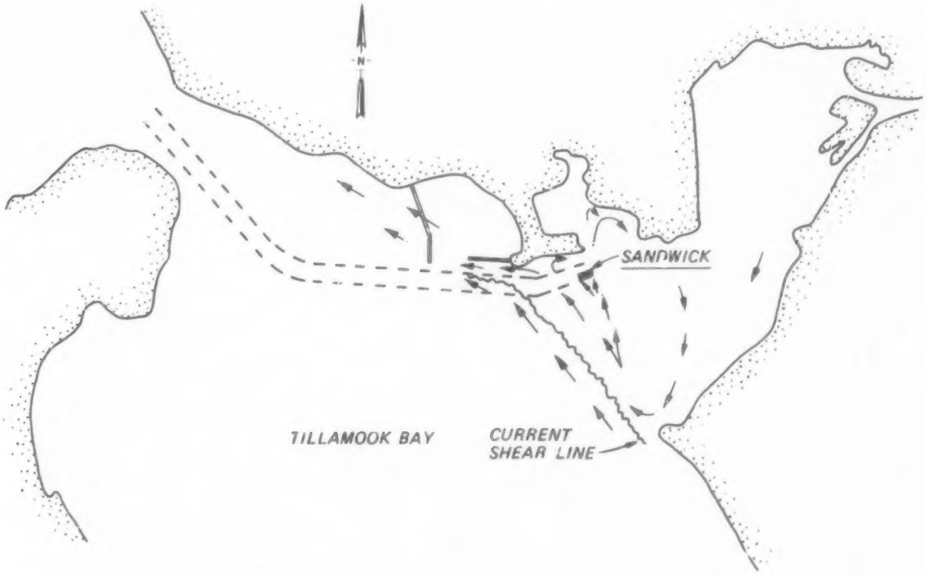


Figure A37. Current patterns during agitation dredging, Tillamook Bay

situation discussed previously for the Delaware Estuary, where such transport was prevented by "null points."

53. Sediment studies showed no resolvable changes in median grain size, uniformity, volatile solids, and carbonates. A change in size gradation downstream of the dredging site ranging from coarser near the site to finer farther away was anticipated but could not be demonstrated by test results. The general conclusion was that the small amount of sediment agitation accomplished made it difficult to measure such effects.

## Savannah Harbor

54. The term "Savannah Harbor" denotes the lower 21 miles of the Savannah River from its mouth to a point approximately 6 miles upstream of Savannah, Georgia, plus an additional 11 miles of entrance channel from the river mouth to the approximate 40-ft-depth contour in the Atlantic Ocean (Figure A38). The river in this area is a partially mixed estuary, with the upstream limit of salinity intrusion and the degree of salt/fresh water mixing depending upon freshwater discharges and tidal stages. Surface currents are predominantly downstream at all locations. However, moving toward the river mouth, bottom currents change from predominantly downstream to predominantly upstream in the vicinity of the lower tip of Hutchinson Island. The resulting bottom current nodal point, similar to those described earlier for the Delaware Estuary, has

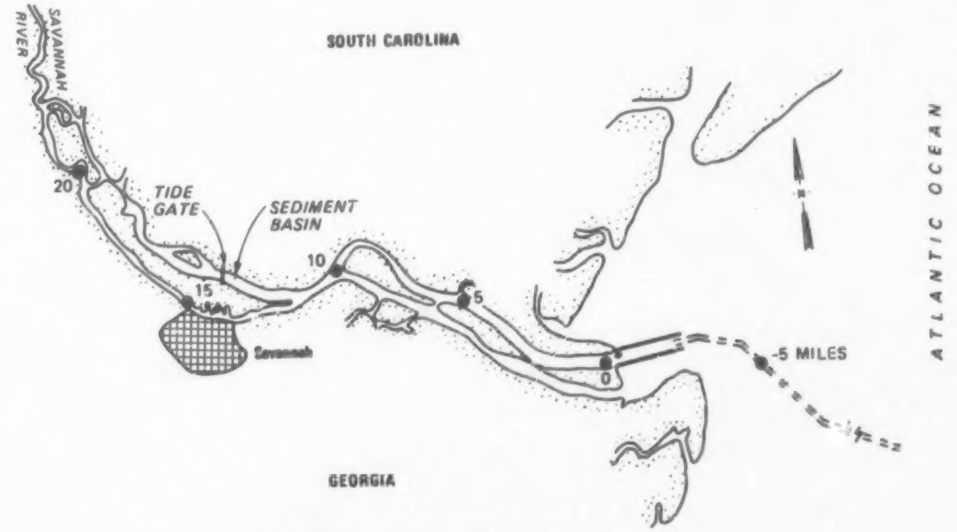


Figure A38. Savannah Harbor

been responsible for heavy shoaling in the lower 4 miles of the Front River (Ippen 1966, USAEWES 1961). This concentrated shoaling led to the construction of a sediment basin and tide gate in the Back River, as shown in Figure A38. The gate, which was placed in operation in 1977, is opened during flood tide and closed during ebb, resulting in sediment deposition in the basin and increased ebb velocities in the Front River.

55. A number of privately owned wharves and docking slips are located along the banks of Savannah Harbor, especially in the Front River region. Many of these features are configured such that they serve as excellent settling basins for fine-grained sediments. Shoaling rates up to 3.5 ft per month have been experienced in some locations, causing serious maintenance problems (Stuber 1976). Historically, a number of these wharves and slips have been maintained by agitation dredging, paid for and conducted by the companies that own them. Agitation dredging is usually accomplished by a tug dragging a steel beam or similar device through the area during ebb tide, the idea being to resuspend settled material so that it is carried into the main channel. Some propeller wash agitation by small craft is also reported by Hussey, Gay, and Bell, Inc. (1975). Since this agitation dredging theoretically increases shoaling volumes in the main channel, the US Army Engineer District, Savannah, assesses a charge for beam dragging operations. In 1973, this charge was $176.00 for each hour of dragging time (Hussey, Gay, and Bell, Inc. 1975).

56. In 1975, the Savannah Harbor Committee on Agitation Dredging, composed of representatives of 13 companies that use agitation dredging to maintain their facilities funded a study of agitation dredging's environmental impact in the harbor. The study also investigated alternatives to agitation dredging that might be environmentally preferable. Results of this study, reported by Hussey, Gay, and Bell, Inc. (1975) and Stuber (1976), are the basis for the remainder of this section.

57. Table A3 lists organizations performing agitation dredging in Savannah Harbor during the period of the environmental study, as well as information describing dredging areas and operations. Figure A39 shows each dredging location in Savannah Harbor. In Table A3 under the heading "Type of Berth," "parallel" refers to a berth with long axis parallel to the navigation channel, while "slip" denotes a berth made by a cut into the riverbank with long axis perpendicular to the channel.

58. Although many of the figures in Table A3 are only estimates, they

## Table A3

## Summary of Agitation Dredging in Savannah Harbor in 1973

| | | | Agitation Dredging Operations | | | | | Yearly Dredging Costs | |
|---|---|---|---|---|---|---|---|---|---|
| Company/Facility Name | Type of Berth | Design Depth ft | No./yr | Maximum cu yd/ Operation | Average Excavation ft | Berth Dimensions ft × ft | Yearly Excavation cu yd | $/cu yd | $ |
| Standard Oil | Parallel | 31 | 2 | 10,700 | 4 | 723 × 100 | 18,400 | 0.13 | 2,400 |
| Union Oil of California | ↓ | 34 | 2 | 7,200 | 2 | 650 × 150 | 14,400 | 0.13 | 1,100 |
| Flintkote | | 35 | 7 | 2,800 | 3 | 1,000 × 25 | NA | NA | NA |
| Swann Oil | | 38 | 0 | -- | -- | 850 × 125 | -- | -- | -- |
| East Coast Terminal | | 32 | 1 | 27,800 | 3 | 2,500 × 100 | 27,800 | 0.13 | 3,600 |
| ITC, Inc. | Slip | 28 | 17 | 37,800 | 3 | 1,700 × 200 | 224,700 | 0.076 | 17,100 |
| Charter Terminal | Slip | 31 | 17 | 15,300 | 3 | 550 × 250 | 211,900 | 0.076 | 16,100 |
| Georgia Ports Authority - Ocean Terminals | | | | | | | | | |
| Berths 1 and 2 | Parallel | 32 | 1 | 9,800 | 2 | 1,100 × 120 | 9,800 | 0.13 | 1,400 |
| Slip 1 | Slip | 26 | 17 | 16,700 | 2 | 1,000 × 225 | 116,600 | 0.13 | 15,400 |
| Slip 2 | Slip | 32 | 17 | 24,000 | 2 | 1,100 × 295 | 118,700 | 0.13 | 15,200 |
| Berths 18-20 | Parallel | 32 | 8 | 14,000 | 2 | 1,600 × 120 | 39,200 | 0.13 | 5,100 |
| Colonial Oil | Parallel | 35 | 8 | 5,000 | 2 | 680 × 100 | 35,100 | 0.25 | 8,800 |
| Savannah Machine and Shipyard | Parallel | 28 | 1 | 29,700 | 8 | 1,000 × 100 | 29,700 | 0.13 | 3,800 |
| | Slip | 20 | 1 | 3,900 | 2 | 350 × 150 | 3,900 | ↓ | 500 |
| Amoco Oil | Parallel | 32 | 7 | 4,400 | 2 | 800 × 75 | 26,700 | | 3,500 |
| Georgia Ports Authority - Garden City Terminals | ↓ | | | | | | | | |
| Berths 50-57 | | 32 | 11 | 57,000 | 2 | 6,400 × 120 | 50,500 | | 6,600 |
| Berth 58 | | 38 | | | | | | | |
| Berth 59 | | 40 | | | | | | | |
| Berths 61-62 | | 36 | 1 | 14,200 | 2 | 1,600 × 120 | 14,200 | | 1,900 |
| Chevron | | 30 | 0 | -- | -- | 700 × 150 | -- | -- | -- |
| Savannah Sugar | | 30 | 0 | -- | -- | 600 × 100 | -- | -- | -- |

(From Hussey, Gay, and Bell, Inc. (1975)).

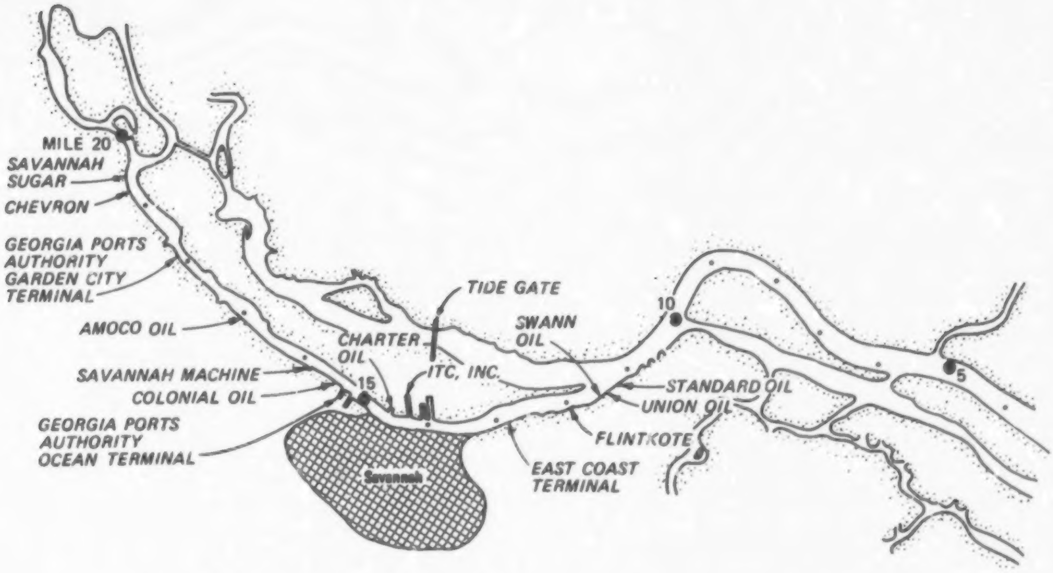

MILE 20
SAVANNAH SUGAR
CHEVRON
GEORGIA PORTS AUTHORITY GARDEN CITY TERMINAL
AMOCO OIL
SAVANNAH MACHINE
COLONIAL OIL
GEORGIA PORTS AUTHORITY OCEAN TERMINAL
Savannah
15
CHARTER OIL
TIDE GATE
ITC, INC.
SWANN OIL
10
STANDARD OIL
UNION OIL
FLINTKOTE
EAST COAST TERMINAL
5

Figure A39. Agitation dredging sites, Savannah Harbor

indicate relative magnitudes of several aspects of Savannah Harbor agitation dredging. First, the total yearly amount of agitation dredging is substantial. The yearly excavations in Table A3 add up to more than 940,000 cu yd, with four locations (ITC, Charter Terminal, and Georgia Ports Authority Slips 1 and 2) contributing over 70 percent of this total. Second, the maximum amounts of material removed in one agitation operation and the average depths of excavation are relatively modest. This may reflect a need to constantly maintain depths at or near design levels as well as a characteristic of the agitation dredging method used. In other words, if a berth shoals 2 or 3 ft, it may have to be dredged then even though the operation will be repeated many times during the year. At the four sites with heavy shoaling described earlier, 17 operations per year were conducted, each removing between 15,000 and 37,000 cu yd and increasing depths 2 to 3 ft. Third, the costs per cubic yard dredged are very small and appear independent of the amounts dredged. The latter observation may result from the fact that mobilization and demobilization costs for the type of agitation dredging performed are minimal compared with those for a conventional hydraulic dredge. In fact, most site-specific costs would be lowered or eliminated in comparison with a hydraulic dredge.

59. The final point to be drawn from Table A3 is that agitation

dredging in Savannah Harbor must be relatively successful from the standpoint of dredging results. It is used at a number of sites by a number of different organizations, moving substantial amounts of material at low costs. Perhaps one additional reason for its popularity is the ease with which the system used in Savannah Harbor can be implemented when needed. Support for this final point comes from Hussey, Gay, and Bell's (1975) study of alternatives to agitation dredging in Savannah Harbor. A number of equipment alternatives ranging from conventional to exotic were investigated, but none were more cost-effective than agitation dredging.

60. For the study commissioned by the Savannah Harbor Committee on Agitation Dredging, three of the berthing sites listed in Table A3 were selected for environmental monitoring: Colonial Oil, Georgia Ports Authority Ocean Terminal Slip 1, and ITC, Inc. Plan views of these sites are shown in Figures A40-A42. Agitation dredging was performed at all three sites by tugboats dragging beams. Surveys made before and after each dragging operation were used to determine the total amount of material removed. The number of dragging hours per operation was recorded, allowing calculation of an average hourly dredging rate for each operation. These data are summarized in Table A4.

61. An obvious fact from Table A4 is that removal rates were much lower at the Colonial Oil site than at the other two. This was attributed to a thinner layer of loosely consolidated silt overlying a much harder bottom at the design depth of 35 ft, and to the fact that this silt layer sloped

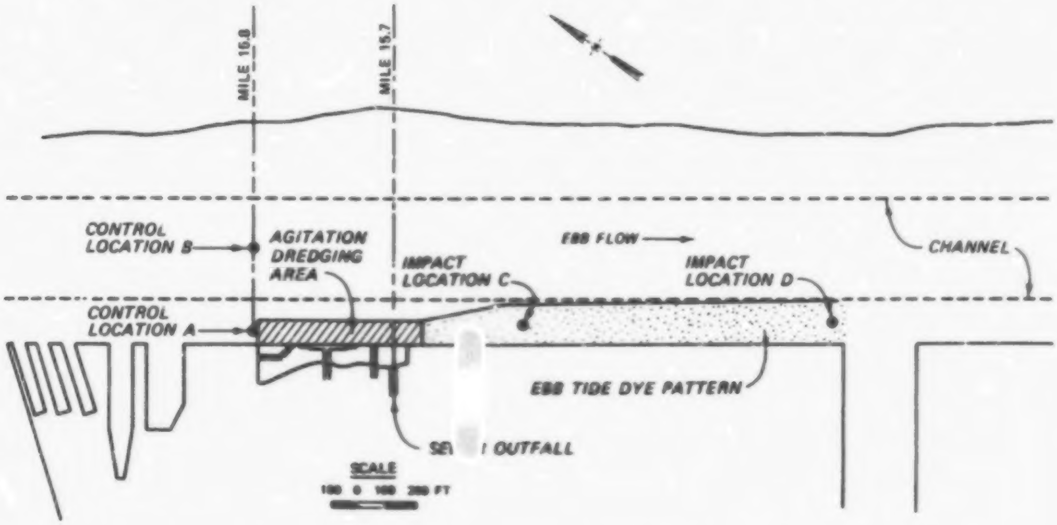


Figure A40. Colonial Oil agitation dredging and monitoring sites

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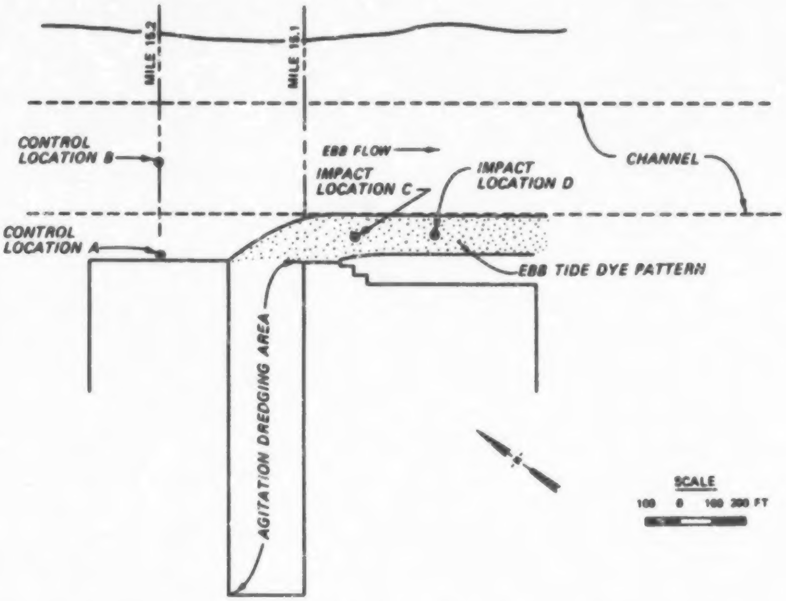
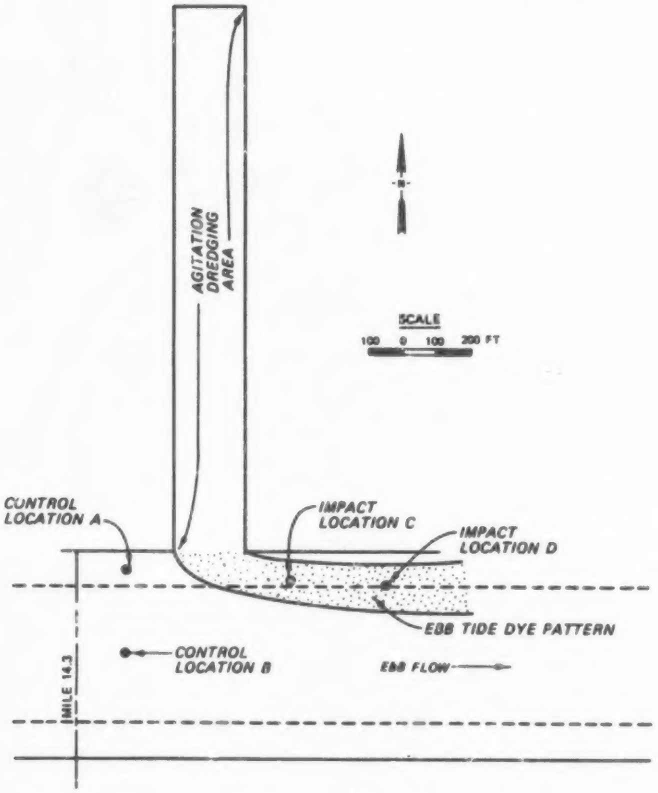

MILE 15.2
MILE 15.1
CONTROL LOCATION B
EBB FLOW
IMPACT LOCATION C
IMPACT LOCATION D
CHANNEL
CONTROL LOCATION A
EBB TIDE DYE PATTERN
AGITATION DREDGING AREA
N
SCALE
100 0 100 200 FT

Figure A41. Georgia Ports Authority Ocean Terminal Slip 1 agitation dredging and monitoring sites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AGITATION DREDGING AREA
SCALE
100 0 100 200 FT
CONTROL LOCATION A
IMPACT LOCATION C
IMPACT LOCATION D
EBB TIDE DYE PATTERN
CONTROL LOCATION B
EBB FLOW
MILE 14.3

Figure A42. ITC, Inc., agitation dredging and monitoring sites

## Table A4

## Results of Monitored Agitation Dredging Operations

| Location | Date | Amount Removed cu yd | Dragging Time hr | Average Removal Rate cu yd/hr |
|---|---|---|---|---|
| Colonial Oil | 5/23/75 | 4,400 | 6.1 | 720 |
| Colonial Oil | 9/3/75 | 800 | 3.3 | 240 |
| Colonial Oil | 10/17/75 | 3,400 | 3.5 | 970 |
| Georgia Ports Authority Ocean Terminals | | | | |
| Slip 1 | 6/18/75 | 8,400 | 4.1 | 2,050 |
| Slip 1 | 8/20/75 | 5,000 | 2.2 | 2,270 |
| Slip 1 | 11/4/75 | 8,600 | 3.2 | 2,690 |
| ITC, Inc. | 8/21/75 | 8,600 | 3.4 | 2,530 |
| ITC, Inc. | 9/4/75 | 2,800 | 2.3 | 1,220 |
| ITC, Inc. | 11/5/75 | 7,300 | 2.2 | 3,320 |

transverse to the direction of dragging. Also, design depths were less at the two sites with higher removal rates. In summary, conditions at these sites were more favorable for a dragging type of agitation dredging than at the Colonial Oil site.

62. A number of environmental parameters were measured at each of the three test sites. During dragging operations, water samples were taken at two control stations upstream of the test site and two impact stations in the path of ebb flow from the site. The impact station closest to the test site was monitored continuously, while the other three stations were sampled intermittently. Water samples were analyzed for suspended solids, turbidity, BOD, COD, ammonia nitrogen, and chlorides. Approximate locations of these stations are shown in Figures A40-A42. The ebb flow path was determined by tracing movement of dye injected into near-bottom water at each site.

63. In addition to water quality monitoring, sediments were sampled in the dredging area and river channel before and after dredging operations. These samples were analyzed for biological and chemical content, and elutriate tests were performed on some. Description of the sediment physical characteristics was limited to in situ density measurements.

64. Table A5 summarizes in a general manner measured values of suspended

## Table A5

## Representative Suspended Solids and Turbidity Measurements

| Location | Suspended Solids mg/ℓ | | Turbidity, JTU | | Remarks |
|---|---|---|---|---|---|
| | Control | Impact | Control | Impact | |
| Colonial Oil | | | | | Decrease in 30-ft impact values noted with decreased removal rate. Impact values decreased markedly with distance from dredging location |
| Near surface | 20-30 | 50-70 | 60 | 80 | |
| 15-ft depth | 40 | 80 | 60-100 | 140 | |
| 30-ft depth | 60-100 | 250-380 | 60 | 300-460 | |
| Georgia Ports Authority Ocean Terminals Slip 1 | | | | | Impact values decreased markedly with distance from dredging location |
| Near surface | 30 | 50 | 60 | 80 | |
| 15-ft depth | 30 | 110 | 60 | 180 | |
| 30-ft depth | 60 | 240 | 120 | 270 | |
| ITC, Inc. | | | | | Near surface impact values decreased to almost control levels at sta D. During second dredging occasion, when removal rate was 1/2 that of other occasions, impact values were less than control levels |
| Near surface | 20 | 100 | 20 | 120 | |
| 15-ft depth | 20-30 | 160 | -- | -- | |
| 30-ft depth | 20-40 | 280-300 | 40 | 320 | |

solids and turbidity at each test site for all tests at that site. The values given in Table A5 depict only representative quantities and trends. Actual levels of the parameters varied considerably with time. The obvious conclusions drawn from Table A5 are that agitation dredging increased both suspended solids and turbidity in the water column, mostly in the near-bottom zone. At each location, the magnitude of this increase fell off sharply with distance from the dredging site. Tests at the Colonial Oil and ITC, Inc., locations indicated a significant decrease in both parameters when the average removal rate decreased.

65. On a number of occasions, impact values of suspended solids and turbidity were observed to increase with time during dragging operations. An example is shown in Figure A43, which plots suspended solids values versus time at three water depths for the 8/21/75 test at ITC, Inc. The increase with

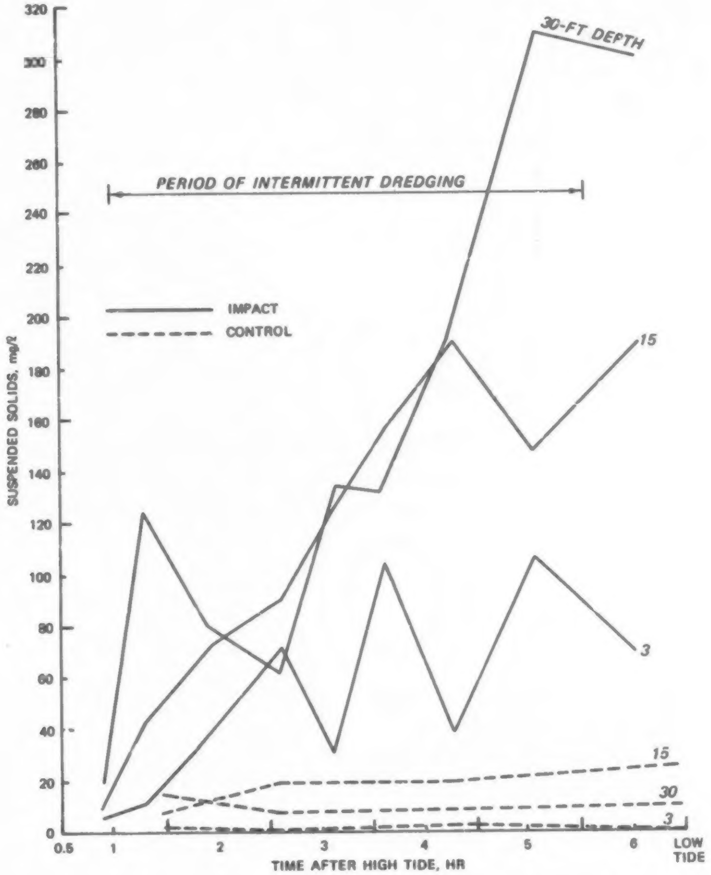


Figure A43. Suspended solids versus time, 8/21/75 test, ITC, Inc.

time of suspended solids at all depth levels is obvious, as is the predominance of suspended solids in the lower half of the water column. Control levels of suspended solids remained consistently low during the entire test. One thing such an increase may indicate is that material was being resuspended in the dredging area faster than it could be carried away by ebb flow, at least during the early part of the ebb tidal cycle. This thesis is indirectly supported by examining the potential mechanisms of ebb flow out of the ITC, Inc., slip.

66. The most obvious ebb flow mechanism is that driven by the falling tide level itself. Its importance in removing sediment resuspended by agitation dredging is questionable, however, since the average tidal prism in the slip is only 25 percent or less of the total water volume in the slip at mean tide level. Due to the rectangular shape of the slip, average currents generated by this prism exiting during ebb flow would be very small. Near the back of the slip especially, tidal flow could probably be discounted as a primary means of removing resuspended sediment.

67. A less obvious but probably more important ebb flow mechanism might be water exchange between slip and river driven by density differences. Krone (1972) presents the results of field measurements of currents, suspended sediments, and salinities at three stations in Savannah Harbor, two of which "bracket" the ITC site (Figure A44). Figure A45 shows measured salinities at these two stations over a spring tidal cycle, which also occurred during the 8/21/75 test at ITC, Inc. Some very rough calculations of water exchange and velocities in the ITC slip due to the change in salinity (and therefore density) of water in the harbor can be made by assuming that salinity in the

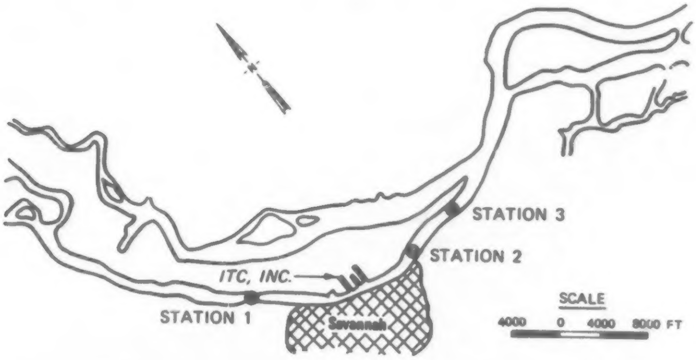
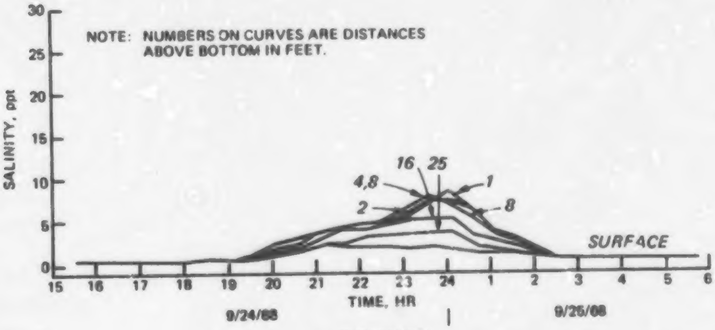
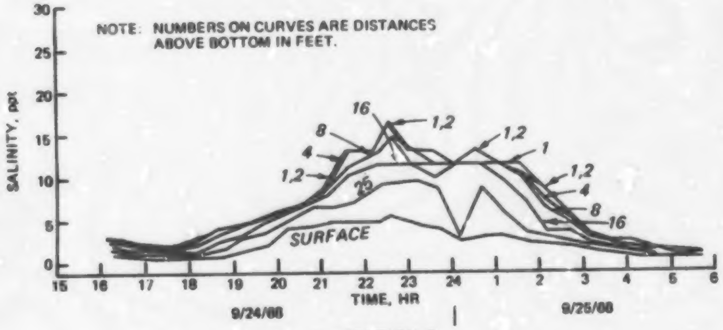


Figure A44. Sampling stations, Savannah Harbor
(Krone 1972)



STATION 1



STATION 2

Figure A45. Salinities, sta 1 and 2, spring tidal cycle, Savannah Harbor (Krone 1972)

harbor changed with time as shown in Figure A46. Under these conditions, water in the slip on a rising tide would be completely exchanged with water from the main channel in approximately 1 hr after salinity in the main channel

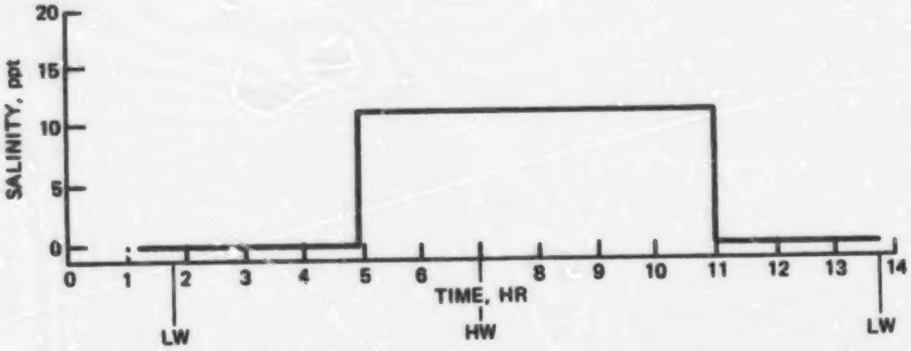


Figure A46. Schematized salinity in main channel at ITC slip (constant with depth)

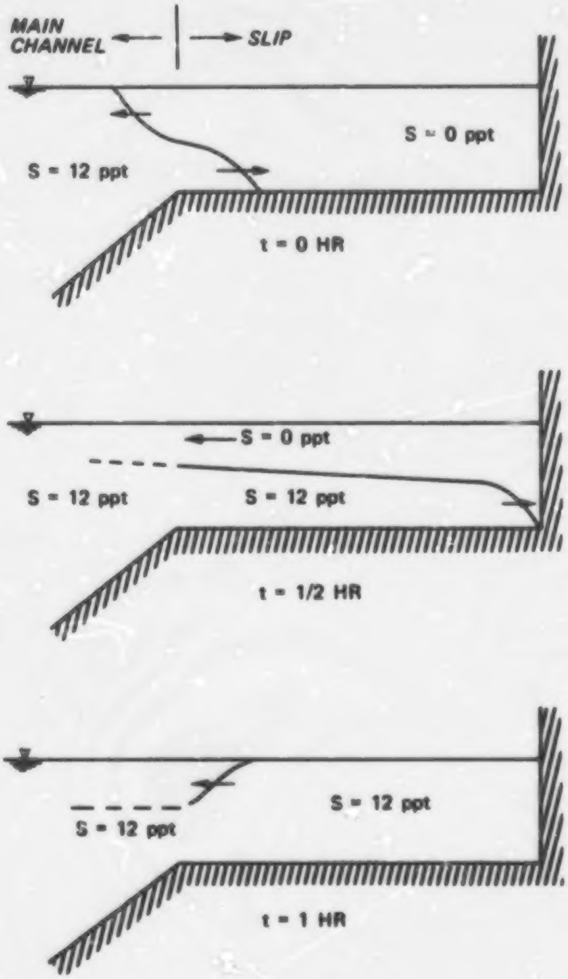


Figure A47. Schematic of slip water exchange by density currents (reversed when main channel salinity returns to 0 ppt)

changed from 0 to 12 ppt. Once this exchange took place, no further density-driven circulation would occur until salinity in the main channel dropped back to zero, 4 hr after high water. Then, the slip water would again exchange completely with main channel water in 1 hr or so. The approximate nature of these exchanges is illustrated in Figure A47. Density currents driven by such a phenomenon would average approximately 1 fps, several orders of magnitude greater than currents driven by a rising or falling tide alone. In reality, these density currents would be smaller and filling/emptying times for the slip would be longer, since actual salinities in the main channel rise and fall gradually for several hours. However, this simple example illustrates the relative importance of density currents in such a situation and the need to consider them in scheduling agitation dredging. For example, on a neap tide, Krone (1972) reports very little change in salinity near the ITC site except at the surface, indicating that density currents would be much less evident and, consequently, that agitation dredging might be less successful. Large variations in freshwater inflow to Savannah Harbor could also alter the salinity regime at a certain location and affect agitation operations.

68. A third mechanism for flow out of the ITC slip might be secondary currents, such as eddies, generated by ebb currents in the main channel. Krone (1972) reported surface ebb currents in the main channel of up to 5 fps on a spring tide. Such a current could generate an eddy in the slip mouth

that would act to exchange slip water for main channel water (Figure A48). This mechanism would function over most of the ebb cycle at varying levels of efficiency, whereas density currents might be of shorter duration. Whether such currents exist at the ITC slip is not known, although Hussey, Gay, and Bell, Inc. (1975) did not report them in their brief description of dye tracer tests at the site. If they do exist, they can be an important mechanism for water exchange in small slips such as ITC, Inc.

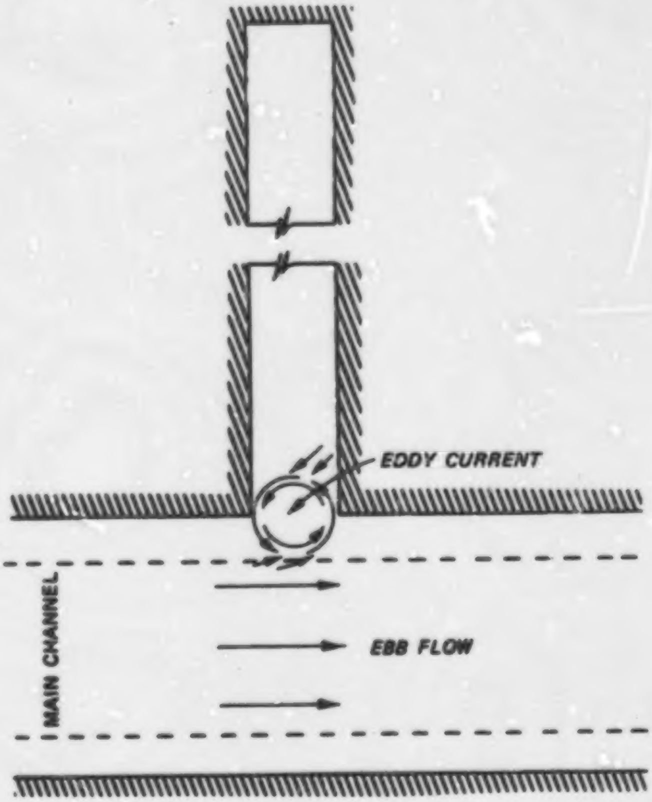


Figure A48. Theoretical eddy current in slip mouth

69. Measurements of effects other than suspended solids and turbidity showed little change due to agitation dredging. D.O. was reduced either not at all or by small amounts. The latter situation became a potential problem on two occasions when background D.O. levels were very low and agitation dredging reduced them to less than acceptable values. (Large variations in background D.O. were observed over the testing program.) COD and ammonia nitrogen increases were on the orders of 50 mg/ℓ and 0.2 mg/ℓ, respectively, although the data were sufficiently varied so that no conclusive trends could be established. Sediments in the dredging areas were determined by Environmental Protection Agency (EPA) standards to be polluted with oxygen demanding organics, grease and oil, and Kjeldahl nitrogen. A composite sample of sediment was reported as having the following characteristics:

| | |
|---|---|
| Density | 70.07 lb/ft$^3$ |
| Total residue | 16.64 percent |

(Continued)

88

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| | |
|---|---|
| COD | 12.40 percent dry weight |
| Volatile residue | 15.92 percent dry weight |
| Ammonia nitrogen | 0.0209 percent dry weight |
| Kjeldahl nitrogen | 0.275 percent dry weight |
| Grease and oil | 0.382 percent dry weight |

70. Any transfer of the above pollutants to the water column appeared to be transient and confined to lower depths. Often, the effects of agitation dredging in this respect were masked by higher levels of pollutants in unconsolidated sediments moved by tidal currents.

71. The effects of drag beam agitation dredging on benthic organisms could not be established in the Savannah Harbor study, since the areas dredged were virtually devoid of such life. Hussey, Gay, and Bell, Inc. (1975) propose several possible reasons for this:

a. Frequent drag beam dredging makes benthic recolonization difficult.

b. Rapid sedimentation limits establishment of a benthic community.

c. Pollutants in sediment are toxic to benthic organisms.

d. Varying salinity levels in harbor water make the region naturally unsuited for benthic organisms.

72. None of the above reasons could be established as a primary cause of benthic conditions.

73. Effects of agitation dredging on motile organisms such as striped bass and on shellfish were projected as insignificant or minimal. Suspended solids levels were not high enough to impair striped bass migration. The other measured effects on water quality were either too small, transient, or localized to be of significance. No studies were made of effects on plankton in the area. No shellfish communities exist in the dredging region, due to channel depth, unsuitable sediments, and periodic low salinities.

## Grays Harbor

74. Grays Harbor is an estuary located on the Pacific coast of Washington, approximately 45 miles north of the Columbia River (Figure A49). The harbor itself is a large bay into which the Chehalis River empties at the eastern end, 15 miles from the ocean. A navigation channel 30 ft deep is maintained from the harbor into the Chehalis River as far as Cosmopolis, where authorized

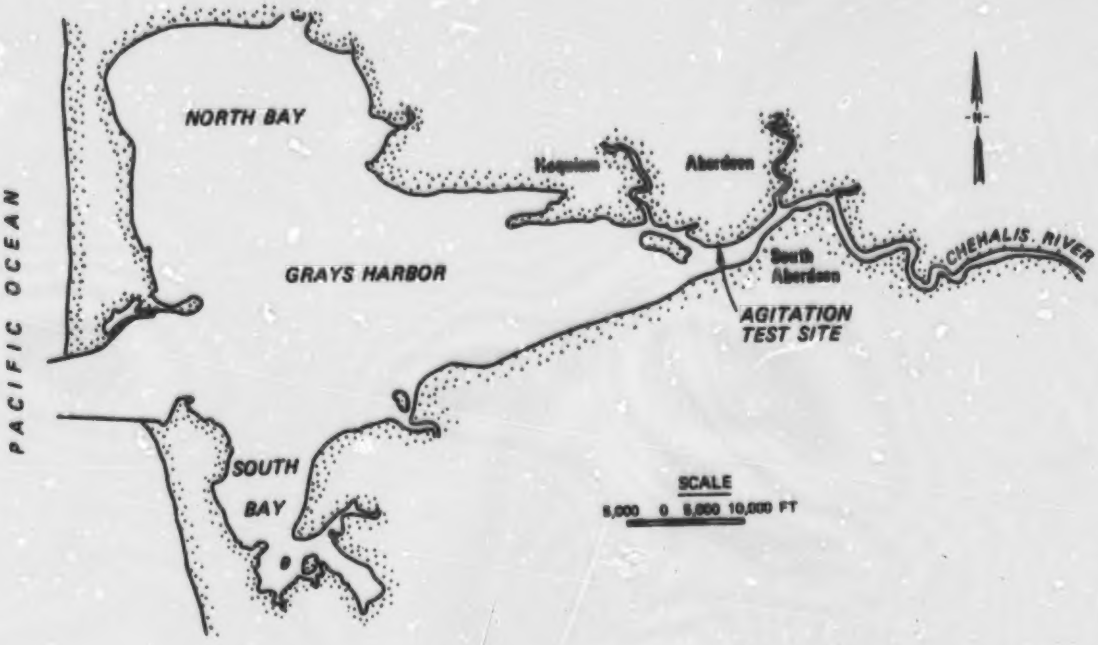
PACIFIC OCEAN
NORTH BAY
Hoquiam
Aberdeen
N
GRAYS HARBOR
South Aberdeen
CHEHALIS RIVER
AGITATION TEST SITE
SOUTH BAY
SCALE
5,000 0 5,000 10,000 FT

Figure A49. Grays Harbor, Washington, agitation test site

depths become 16 ft up to Montesano. The estuary is of the partially mixed type, meaning that a salinity gradient exists in the vertical direction but no distinct saline wedge is evident. The upstream limit of salinity intrusion under average conditions is between the Wishkah River mouth and Junction City. Bottom ebb currents in the vicinity of Aberdeen and South Aberdeen reach values of 1 fps on spring tide (Brogdon 1972).

75. A large dock known as Terminal 4 is located at the junction of Grays Harbor and the Chehalis River as shown in Figure A49. The dock layout is shown in Figure A50. The dock is subject to shoaling underneath, extending into berthing areas for ships using the facility. The average shoal material consists of 62 percent silt, 21 percent clay, and 18 percent sand-size particles (Grays Harbor College 1973).

76. In 1971, 46 aerating and mixing units known as Helixors* were installed on the outer row of bearing piles along a 600-ft stretch of Terminal 4. Details of the installation are shown in Figures A51 and A52. Figure A53 is a detail of a Helixor unit, consisting of a pipe 10 ft long, 18 in. in diameter, with an internal helix running the entire length. Compressed air is injected

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* Manufacturer - Polcon Corp., Montreal, Canada.

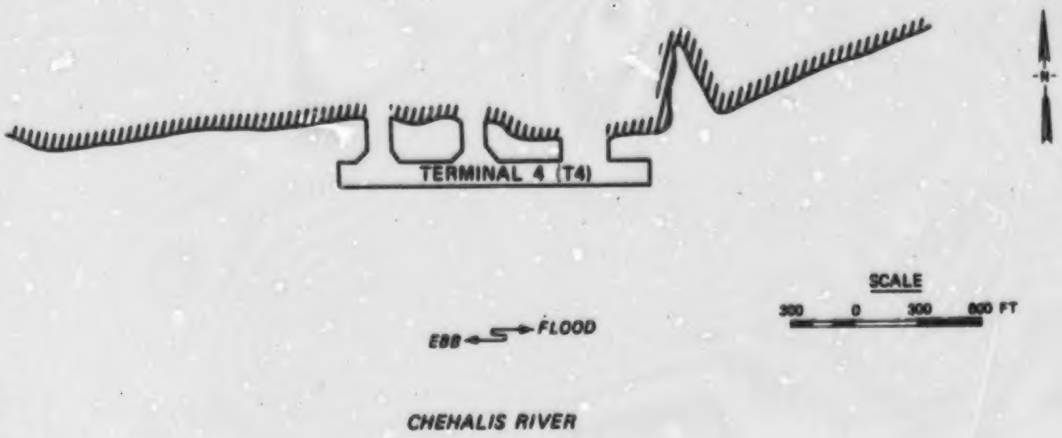
TERMINAL 4 (T4)
N
SCALE
300 0 300 600 FT
EBB
FLOOD
CHEHALIS RIVER

Figure A50. Terminal 4, Grays Harbor

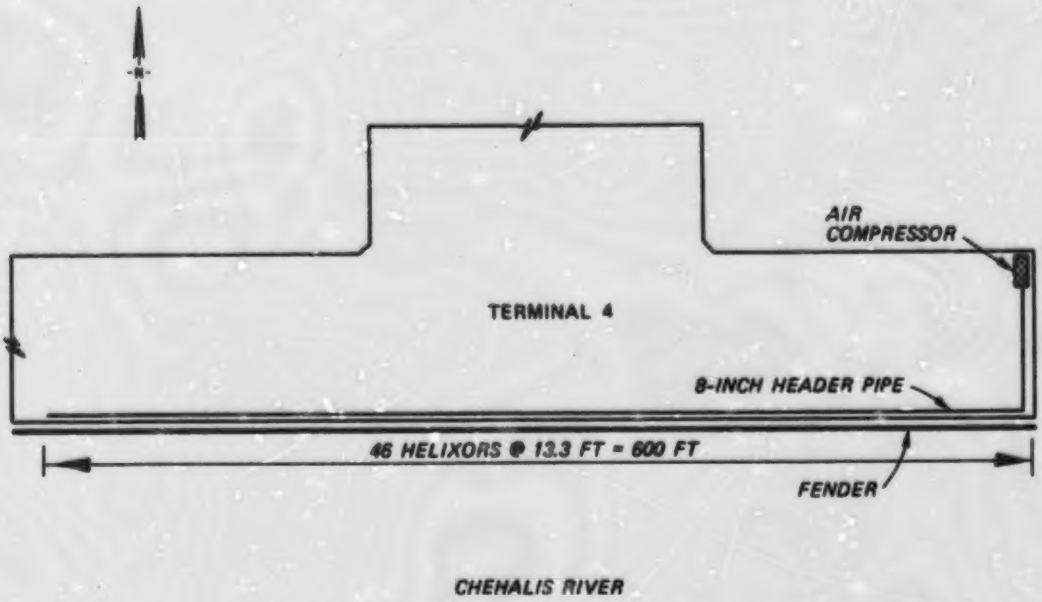
N
AIR COMPRESSOR
TERMINAL 4
8-INCH HEADER PIPE
46 HELIXORS @ 13.3 FT = 600 FT
FENDER
CHEHALIS RIVER

Figure A51. Plan view, Helixor installation, Grays Harbor

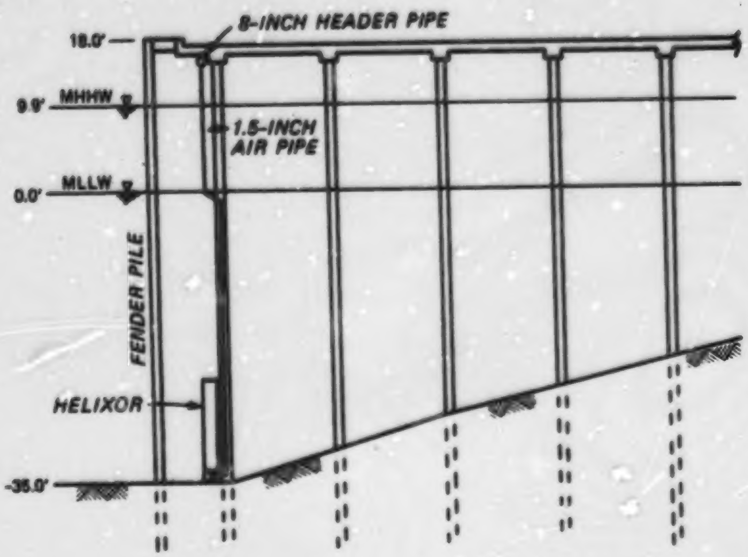
18.0'
8-INCH HEADER PIPE
9.9' MHHW
1.5-INCH AIR PIPE
0.0' MLLW
FENDER PILE
HELIXOR
-35.0'

Figure A52. Elevation view, Helixor installation, Grays Harbor

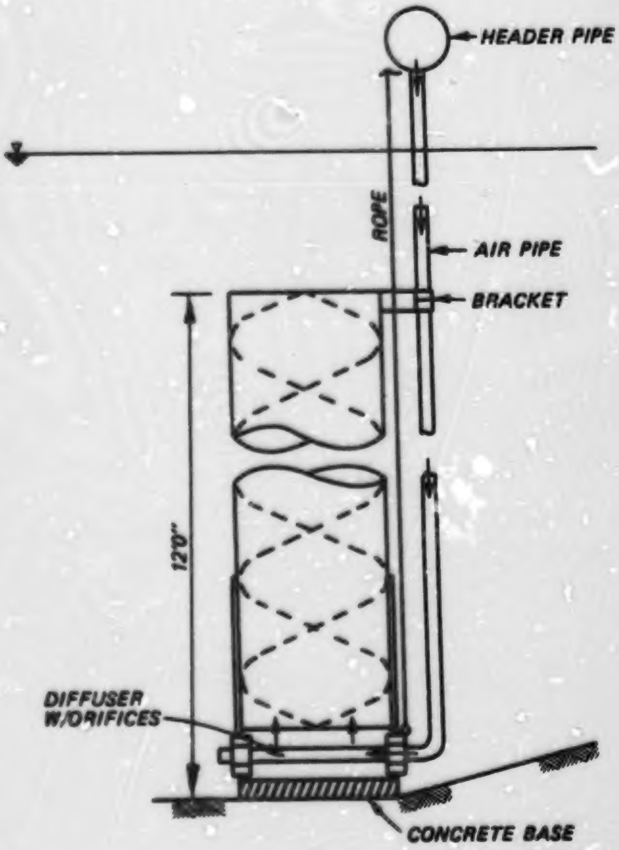
HEADER PIPE
ROPE
AIR PIPE
BRACKET
12'0"
DIFFUSER W/ORIFICES
CONCRETE BASE

Figure A53. Helixor unit

into the open pipe bottom and rises through the pipe, causing near-bottom water and sediment to follow suit. A mixed plume of air, water, and sediment then exits the pipe top at a velocity of 4 fps or more, according to the manufacturer (Noble*). Again, according to the manufacturer (Smolski 1972), such a system would prevent shoaling in the berthing areas by two means:

- a. Maintaining solids in suspension during slack water periods so that they can be carried off by subsequent tidal currents.
- b. Resuspending deposited sediment sliding into the berthing areas from beneath the dock.

77. Each Helixor unit required 39 scfm of compressed air, for a total air requirement of approximately 1,800 scfm. At this supply rate, each unit could pump approximately 3 mgd (2,080 gpm), giving a total throughput of 95,000 gpm (Smolski 1972). Air was supplied to the installation by two compressors through an 8-in. header pipe. Each Helixor unit received air from a 1.5-in. pipe off the 8-in. header.

78. In January through June 1973, a study was conducted to ascertain effectiveness of the Helixor units in preventing sedimentation (Grays Harbor College 1973). In April 1973, while this study was under way, a second aerating and mixing system, called Ventra-Vac, was installed near the Helixor units. This system consisted of two Ventra-Vac units installed approximately as shown in Figure A54. Each unit is configured and operates as shown in Figure A55. Compressed air enters a plenum chamber in the unit body and exits through holes around a circular flared section. Air rises from the flared section through a pipe toward the surface, inducing fluid flow to accompany it. This causes water and solids to be drawn into the Ventra-Vac unit at its cone-shaped bottom and exit near the surface. Thus the principle of operation and anticipated benefits of Ventra-Vac units are similar to those described previously for the Helixor system.

79. The study conducted by Grays Harbor College involved sampling nine stations once a week for the following parameters: suspended sediment, current velocity, D.O., temperature, and salinity. In addition, sediment cores were taken at each station in April and June. Further monitoring was conducted at two to four stations of varying location once per month for a 13-hr period. The study objective was to separate changes in water quality and suspended

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* R. J. Noble (1974); letter to US Army Engineer District, Seattle.

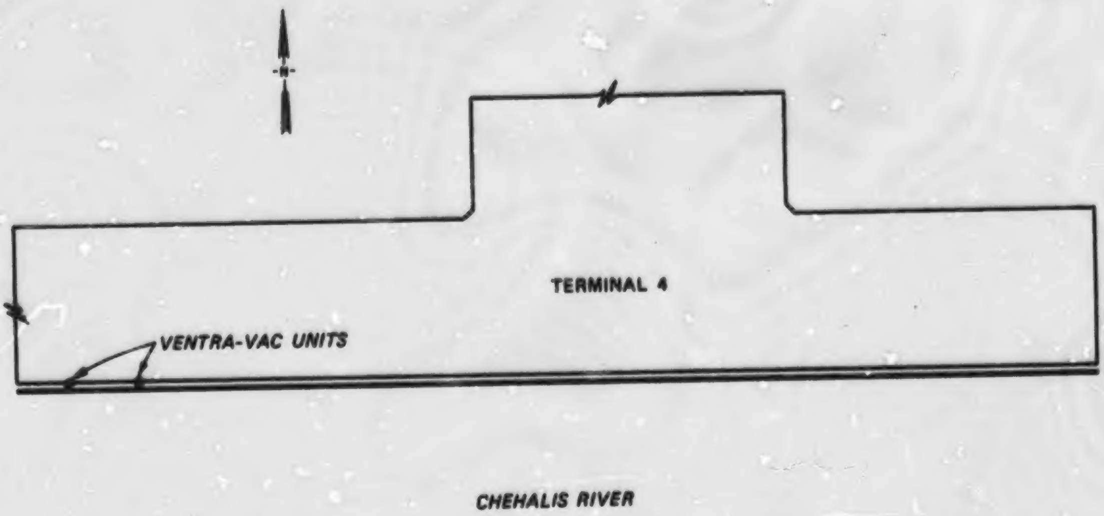
TERMINAL 4
VENTRA-VAC UNITS
CHEHALIS RIVER

Figure A54. Ventra-Vac unit locations, Grays Harbor

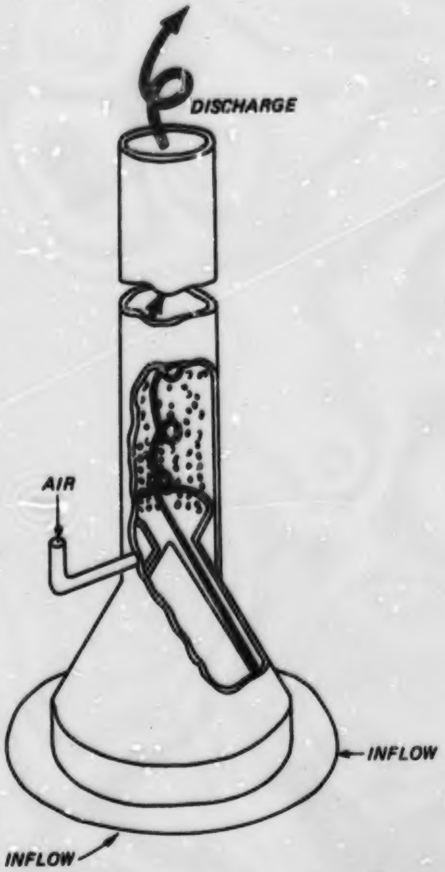
DISCHARGE
AIR
INFLOW
INFLOW

Figure A55. Ventra-Vac unit

sediments caused by the Helixor and Ventra-Vac units from changes occurring naturally in the estuary.

80. Sample station locations are given in Figure A56. Sta 1 and 4 were considered to represent channel center-line conditions, 5 and 6 the channel north side, 8 and 8A at Terminal 4 away from the aerating and mixing systems, 2 and 3 at the Helixor system, and 7 at the Ventra-Vac system (after mid-April).

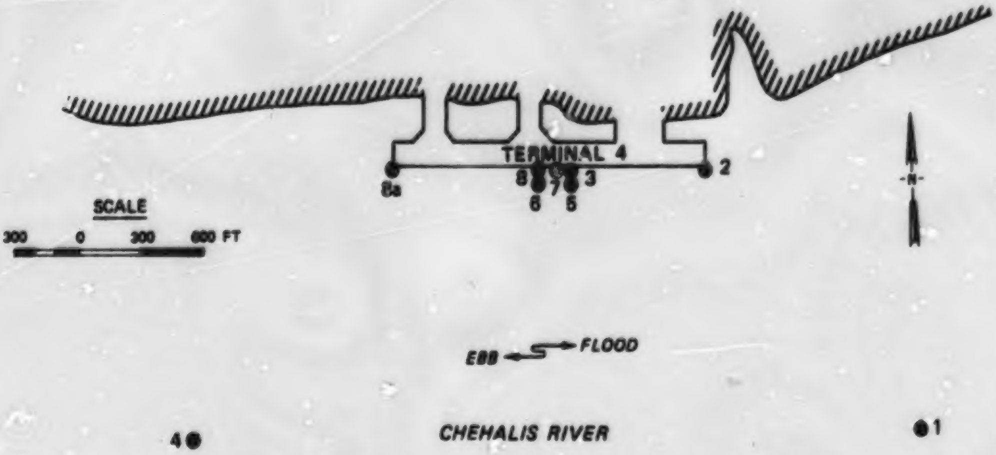

Figure A56. Sample stations, Grays Harbor

81. Study results indicated that both the Helixor and Ventra-Vac systems were effective in raising bottom water to the surface and aerating it at the same time. Normal stratifications of density, salinity, and temperature were broken up in the vicinity of both systems. This effect was very localized, however, as the denser water brought up by the systems sank back to the bottom within 20 ft from Terminal 4. Some recirculation of the water may also occur, as evidenced by the observed vertical currents shown in Figure A57. D.O. was approximately 18 percent higher in water after it was raised by the systems than before. However, this increase also was confined to the immediate vicinity of the systems.

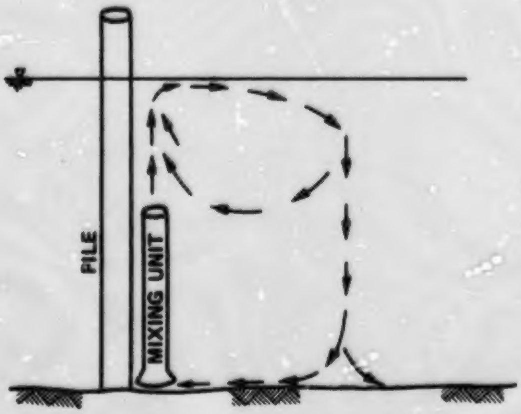

Figure A57. Current patterns generated by mixing units, Grays Harbor

82. The amount of suspended sediment in the zone of influence of both

systems varied considerably, but was generally greater than ambient by an average of 25 mg/ℓ in February and March and 13 mg/ℓ in May. One comparison of grain-size distributions indicated that suspended sediment near the systems was somewhat coarser than suspended sediment away from them. The study report hypothesizes that this phenomenon as well as some variations in suspended sediment amounts are related to variations in near-bottom load brought to the study site by ebb and flood currents, since this near-bottom load would usually be coarser than ambient suspended load.

83. The Terminal 4 region was dredged in December 1972 and March 1973. By comparing the 1972 postdredging and 1973 predredging surveys, it was estimated that 32,000 cu yd of shoaling occurred in the region shown in Figure A58.

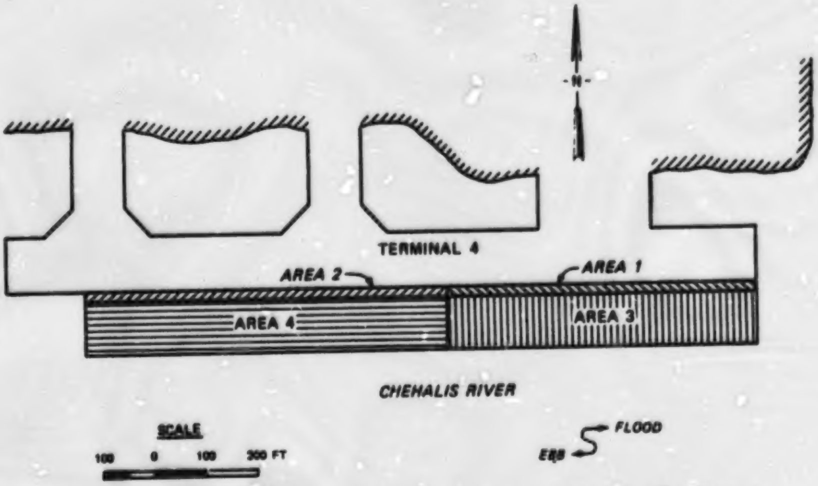

Figure A58. Bottom survey areas, Grays Harbor

Of this total, 2,300 cu yd shoaled in area 1 fronting the Helixor and Ventra-Vac systems out to 20 ft from the terminal. A similar area (2) not influenced by these systems accumulated 3,000 cu yd. However, comparing these areas with corresponding ones (3 and 4) covering 20 to 120 ft out from the terminal, the ratios of shoaling are virtually identical:

|  | Area 1 | Area 3 | Area 2 | Area 4 | 1/3 | 2/4 |
|---|---|---|---|---|---|---|
| Shoaling volumes, cu yd | 2,300 | 11,800 | 3,000 | 15,500 | 0.195 | 0.193 |

84. On this basis, it was not possible to attribute any significant reduction in shoaling at Terminal 4 to the Helixor or Ventra-Vac systems. A number of factors may have been involved in these results:

a. Vertical circulation patterns generated by the systems may create a semiclosed "loop," wherein some suspended sediment is recycled through the systems.

b. Drogue studies indicated that the systems' circulation pattern may reduce local tidal current velocities, possibly encouraging sediment deposition in the vicinity of the systems.

c. Near-bottom currents generated by the systems flow toward the Helixor and Ventra-Vac units. These currents may bring with them some near-bottom load from outside the systems' circulation pattern, adding to the total sediment load within the pattern.

d. The Helixor units, at least, may not have been installed in the manner recommended by the manufacturer. The manufacturer (Noble*) claimed subsequent to the Grays Harbor College study that the Helixor units were to have been installed at a location dredged clear of all silt to -35 ft below mllw and that instead they were placed in a bank of silt 10 to 12 ft deep. He further claimed that difficulties with the system air compressor made the Helixors inoperative for periods of time, allowing additional sediment deposition on the units.

85. The Grays Harbor College study report made little differentiation between performance of the Helixor and Ventra-Vac units. Suspended sediment values were comparable within the bubble streams of both systems, while Ventra-Vac added roughly 2 percent more D.O. to the water than Helixor.

## Mare Island Naval Shipyard

86. Mare Island is located at the eastern end of San Pablo Bay, part of the San Francisco Bay estuarine complex (Figure A59). The Mare Island Naval Shipyard (MINSY) occupies a major part of the island shoreline fronting Mare Island Strait, principal connection between the Napa River and the estuarine complex (Figure A60). Shoaling is a major problem in the MINSY area. The navigation channel and ship turning basin are maintained by hopper dredge at 30-ft depths, while the slip and pier areas are cleaned by hydraulic pipeline dredge. The latter areas are particularly difficult to maintain due to structures and docked vessels. When a vessel is docked for more than a few months, it can become surrounded by shoaling, since the pipeline dredge cannot work underneath it. Dry docks experience similar problems. Since the pipeline dredge cannot work under piers, material in this area often slumps into a

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* See footnote on page A56.

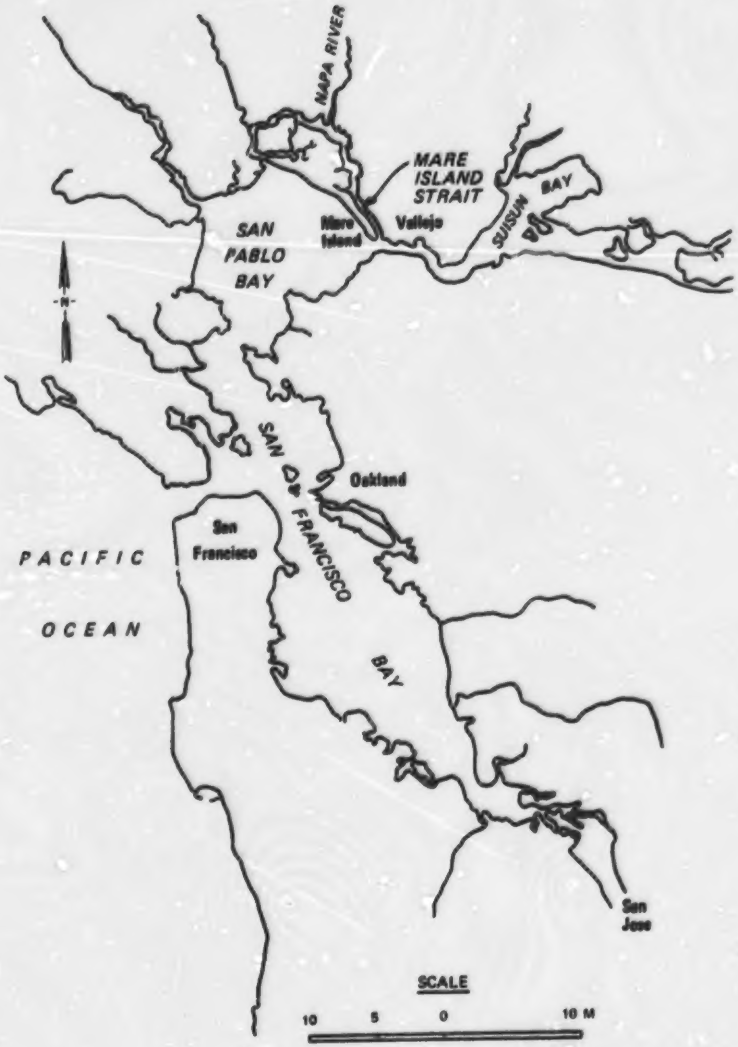

NAPA RIVER
MARE ISLAND STRAIT
SUISUN BAY
SAN PABLO BAY
Mare Island
Vallejo
N
SAN FRANCISCO BAY
Oakland
San Francisco
PACIFIC OCEAN
San Jose
SCALE
10
5
0
10 M

Figure A59. Mare Island and Mare Island Strait

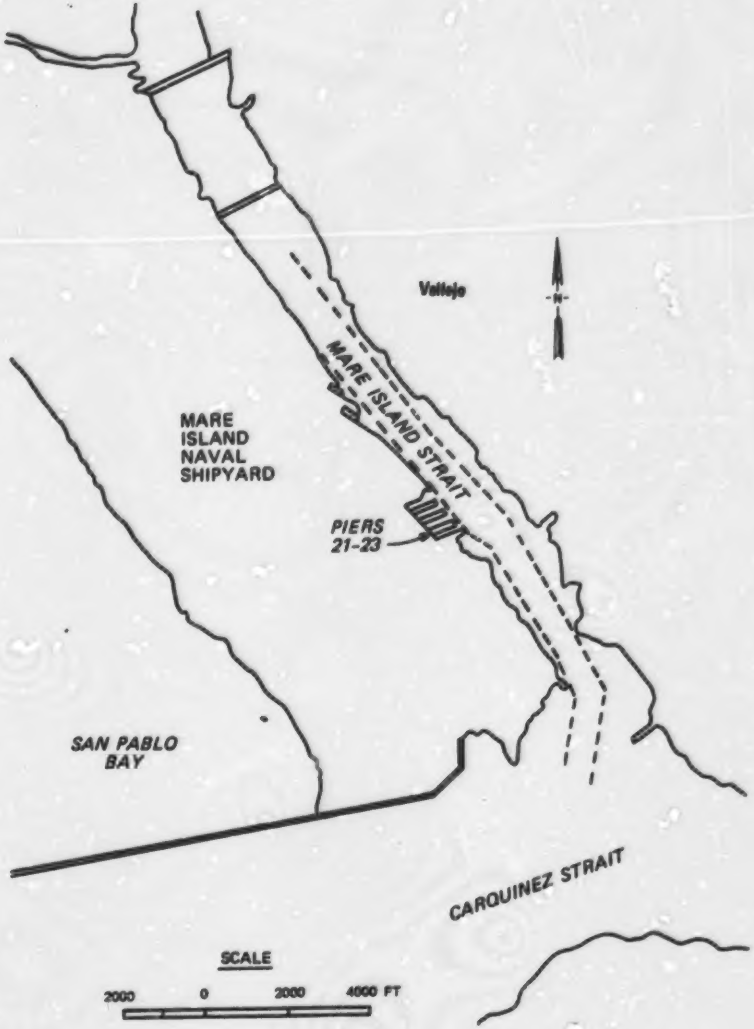

Vallejo
N
MARE ISLAND STRAIT
MARE ISLAND NAVAL SHIPYARD
PIERS 21-23
SAN PABLO BAY
CARQUINEZ STRAIT
SCALE
2000
0
2000
4000 FT

Figure A60. Mare Island Naval Shipyard

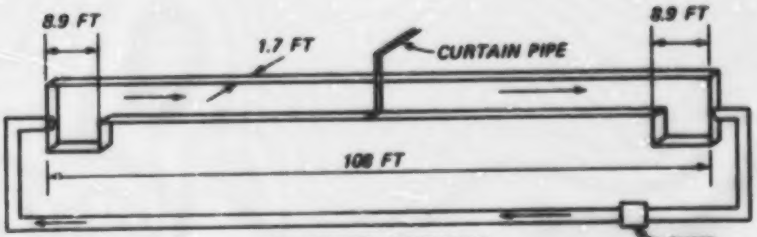
newly dredged cut, causing high shoaling rates immediately after dredging.

87. In 1974, the Navy contracted with the University of California's Scripps Institution of Oceanography (SIO) to study sedimentation problems at Navy facilities, with a particular view toward low-cost alternatives to dredging. Several of these alternatives, studied analytically in the laboratory and as prototype installations, could be classified as agitation dredging.

88. In the period 1975-1981, SIO issued four reports describing progress in the study (Van Dorn, Inman, and Harris 1975; Van Dorn et al. 1977; Van Dorn, Inman, and McElmury 1978; Jenkins, Inman, and Van Dorn 1981). These reports are summarized in the following paragraphs by describing the agitation dredging devices, test results, and data gathered in periodic surveys of local hydrodynamic and sedimentary processes.

89. An early effort in the SIO study was a series of laboratory tests. The first tests were aimed at determining physical and chemical properties of Mare Island bottom sediments and how they changed with time. Samples were taken from 1.0, 3.3, and 6.6 ft below the sediment/water interface, corresponding to depositional ages of 3, 6, and 12 months, respectively. Appearance and composition were the same at each depth. Water comprised 70 percent of sediment weight at a constant salinity of 12 ppt. Solids were 46 percent kaolinite and chlorite, 29 percent illite, and 25 percent montmorillonite, with traces of organics and diatoms. Particle size tests, although inconclusive, indicated sizes in the clay and silt range. Conductivity and viscosity both increased with sample depth. Assuming a "terminal compaction" density of 300 g/ℓ, compaction tests indicated that a freshly settled deposit reached 25 percent of this value in 1 hr and 50 percent in 11.5 hr, the compaction rate varying hyperbolically with time.

90. The next laboratory tests examined models of air and water curtains to hinder sediment intrusion into slip areas. Although such curtains are not agitation dredging devices, model results provided some insight into others that claim to be, such as the Harbour Town Marina system described previously. Both types of curtains, which act by creating an upward turbulent current, were tested in a small flume (Figure A61). Water was pumped through the flume with Mare Island sediment in suspension at a mean velocity of 0.06 fps. With no flow through the air curtain pipe laid across the flume bottom, deposition patterns appeared as shown in Figure A61. Figure A62 shows deposition patterns for low air curtain flow, high air curtain flow, and high air curtain


8.9 FT
1.7 FT
CURTAIN PIPE
8.9 FT
108 FT
PUMP

TEST FLUME SCHEMATIC

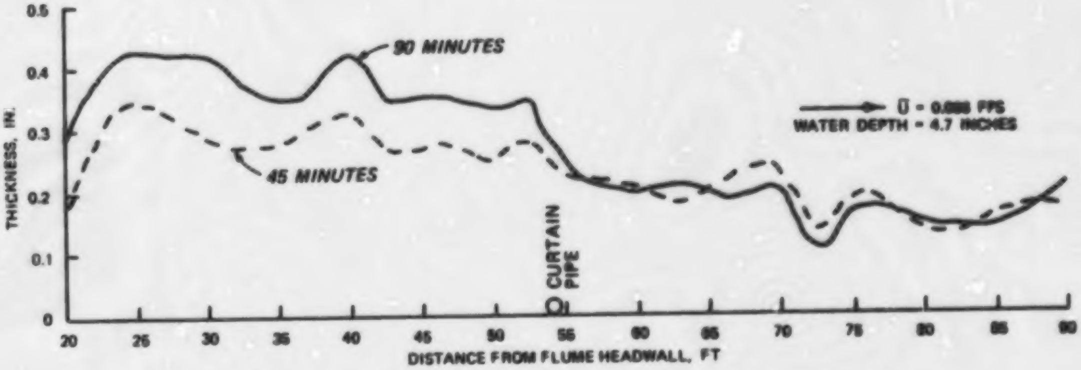


Figure A61. Test flume schematic and deposition patterns, air curtain, no airflow

100

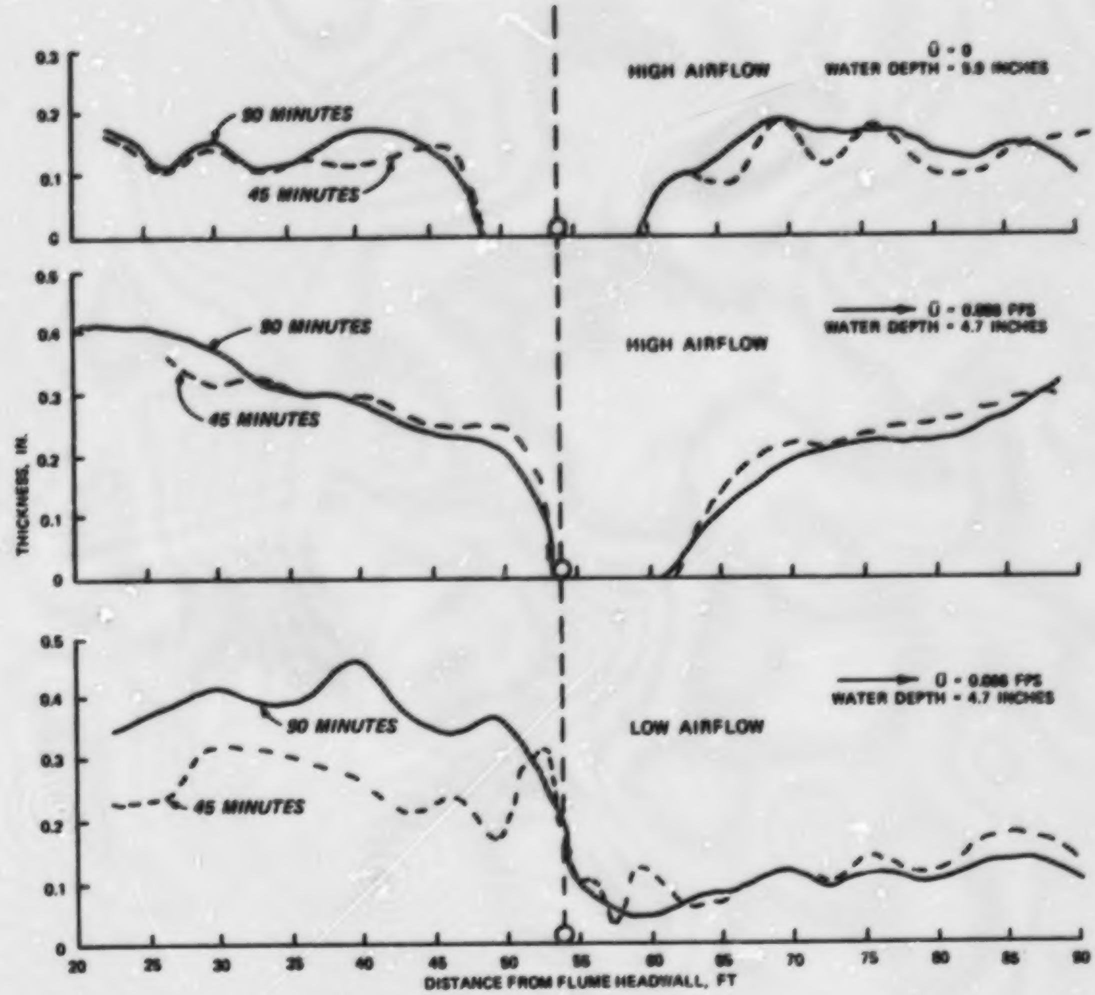


Figure A62. Deposition patterns, air curtain

flow with no flume flow. Figure A63 shows the same for no water curtain flow, high water curtain flow, and high water curtain flow with no flume flow. Several conclusions from the model tests may pertain to similar agitation dredging devices:

- a. Both types of curtains produced zones of hindered deposition (called scour in test report) symmetrical around the curtain pipe with no current in the flume.
- b. Flume current caused hindered deposition zone to shift downstream, making it asymmetrical to curtain pipe. Zone width remained the same.
- c. Low air curtain flow could not prevent deposition completely.
- d. Air curtain was not as effective as water curtain in blocking sediment transport.

001

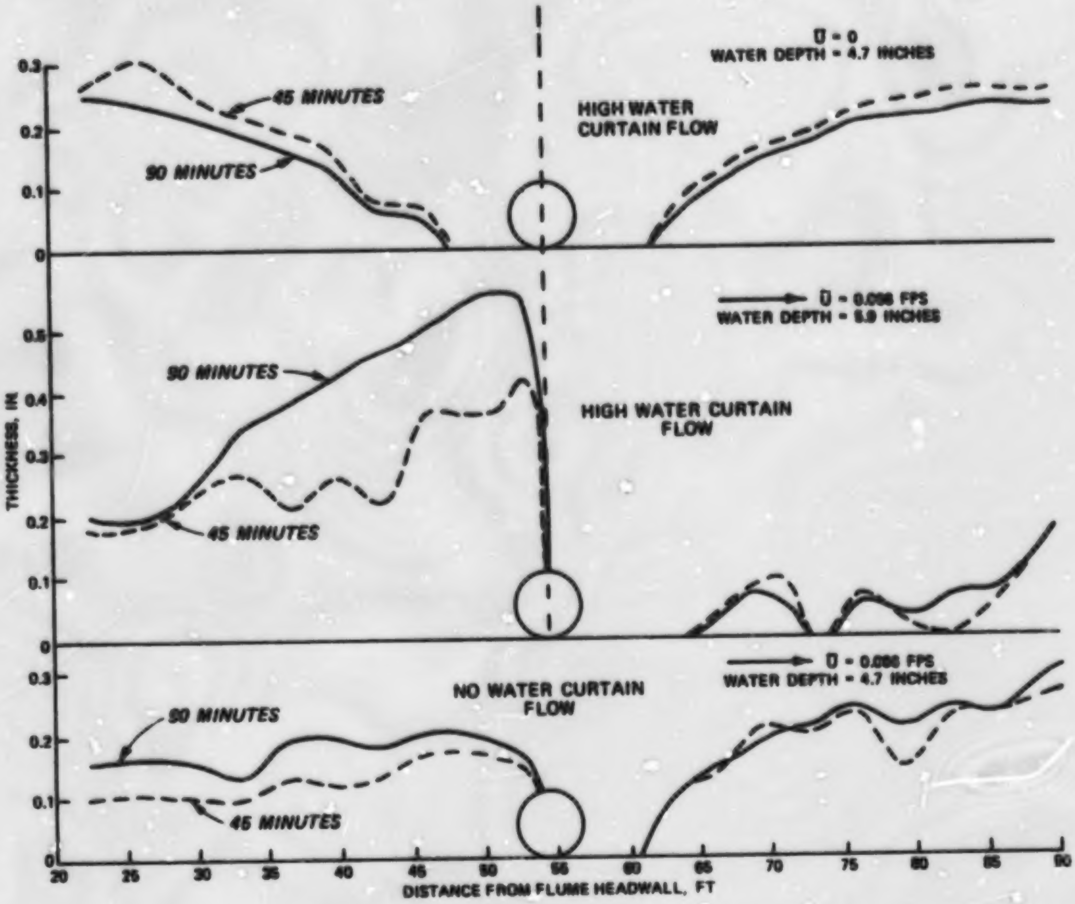


Figure A63. Deposition patterns, water curtain

91. The most extensive laboratory work by SIO was devoted to the hydraulic and sediment scouring characteristics of submerged water jets. The reason for these tests was to develop design criteria for an array of jets to be installed on a slip bottom and operated periodically to resuspend recently deposited sediment. A number of model tests were conducted initially on single jets to determine scour patterns developed by various discharges and configurations. Bottom shear stress measurements were made over the jet pattern at varying discharges. Results were expressed as a relation between bottom shear stress, discharge, and distance from the jet. Conclusions were:

a. The scour pattern limit (boundary) represents an isoline of constant bottom shear stress.

b. The shear value associated with this isoline is approximately $2.1 \times 10^{-3}$ lb/ft$^2$ for newly deposited Mare Island sediment and $5.2 \times 10^{-3}$ lb/ft$^2$ for compacted sediment.

102

92. The next laboratory model tests involved an array of 10 jets with overlapping patterns installed in a 1:100-scale model of the berthing area between two piers at MINSY (Figure A64). By operating the jets two at a time beginning at the berth head and working toward the mouth, the berth area could be cleared of sediment in approximately 5 min. The agitated sediment was carried into what represented the Mare Island navigation channel. By synchronizing such an operation with channel ebb currents, hopefully sediment would be carried away from the berth area. Scaling up results of the 10-jet array model, it was estimated that the prototype berth area could be maintained by a jet array supplied by two 200-hp pumps operated 2 to 3 hr per day.

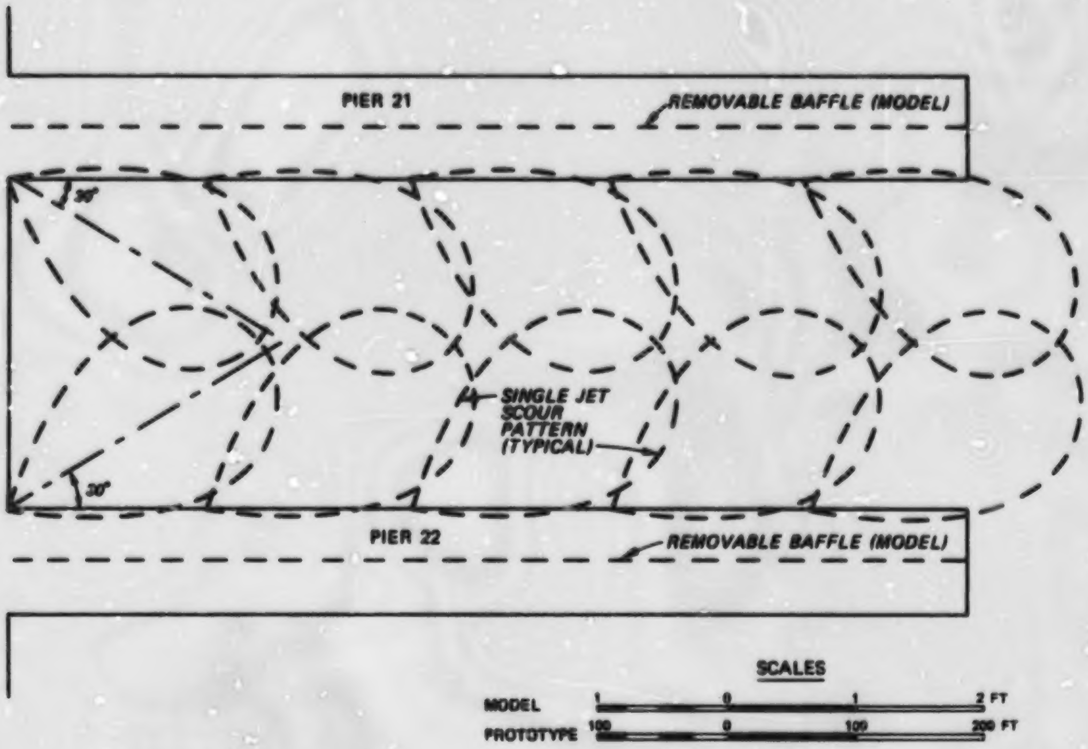

PIER 21
REMOVABLE BAFFLE (MODEL)
30°
30°
SINGLE JET SCOUR PATTERN (TYPICAL)
PIER 22
REMOVABLE BAFFLE (MODEL)
SCALES
MODEL
1
0
1
2 FT
PROTOTYPE
100
0
100
200 FT

Figure A64. Scour jet array model layout, Piers 21 and 22, Mare Island

93. In addition to the 10-jet array, two pipes with a series of small, closely spaced holes were tested as a possible means of producing a comb-shaped scour pattern. The pipes were installed beneath and parallel to the piers, with the holes at 70-deg angles to the pier longitudinal axis. Scouring with this arrangement was deemed not as effective as the 10-jet array, since it covered mainly areas near the pier. However, the installation showed

some possibilities for preventing sedimentation around vessels docked for long time periods and at dry dock entrances.

94. Following initial laboratory tests, a field survey was conducted at Mare Island over a 1-week period in March 1976. The survey purpose was to categorize the Mare Island sedimentary environment and to investigate deposition mechanisms in the berth area and main channel. Survey station locations are shown in Figure A65.

95. Currents at sta A1 over a tidal cycle are shown in Figure A66. The depth distribution shows an apparent flood predominance at lower depths, a situation which could increase shoaling by acting to retain moving sediment within the strait. Maximum ebb currents were 2.6 fps at the surface and 0.6 fps near bottom. Ebb flow began about 1 hr after high tide and lasted 5 hr.

96. Figure A67 shows surface and bottom current roses over a tidal cycle for sta A5 in the mouth of a berthing slip. Bottom flow is generally into the

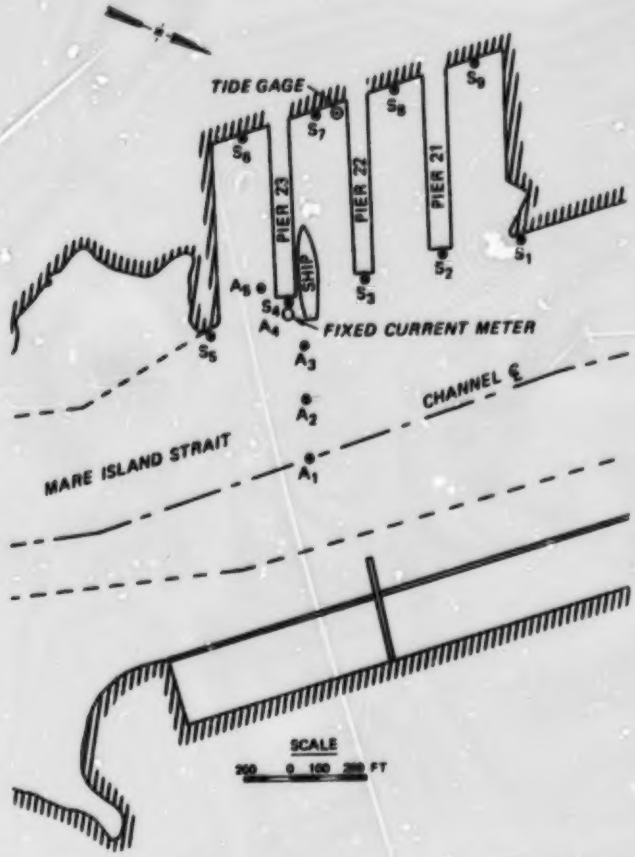

TIDE GAGE
$S_6$
$S_7$
$S_8$
$S_9$
PIER 23
PIER 22
PIER 21
SHIP
$A_5$
$S_4$
$A_4$
FIXED CURRENT METER
$S_3$
$S_2$
$S_1$
$S_5$
$A_3$
$A_2$
$A_1$
CHANNEL ℄
MARE ISLAND STRAIT
SCALE
200 0 100 200 FT

Figure A65. Survey stations, Mare Island

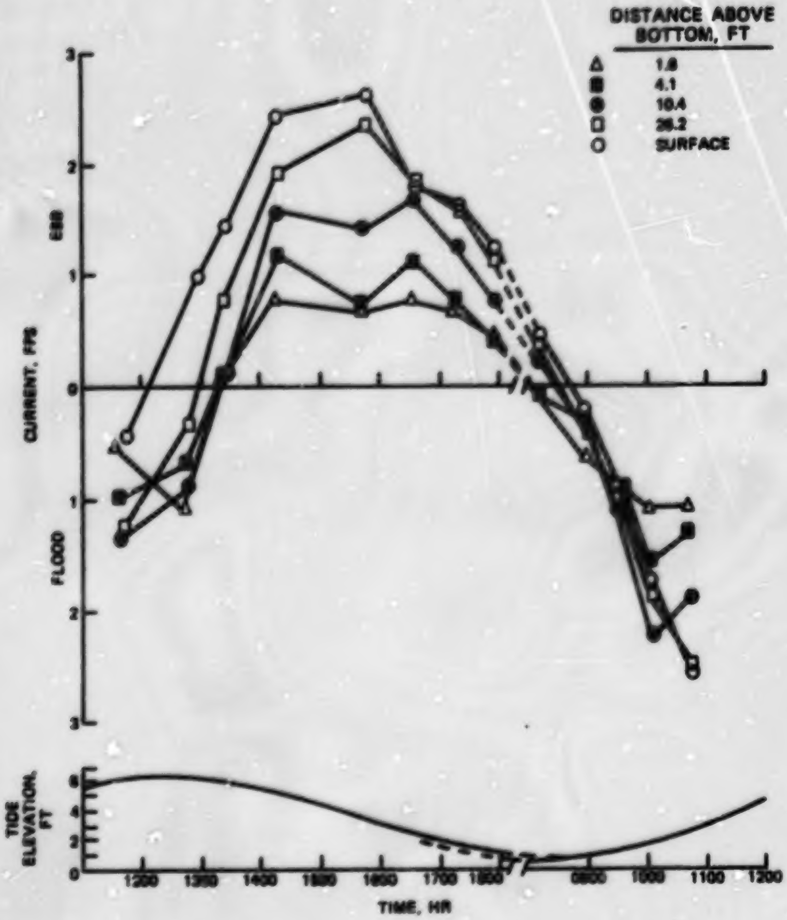


Figure A66. Currents and tide elevation, sta A1, Mare Island

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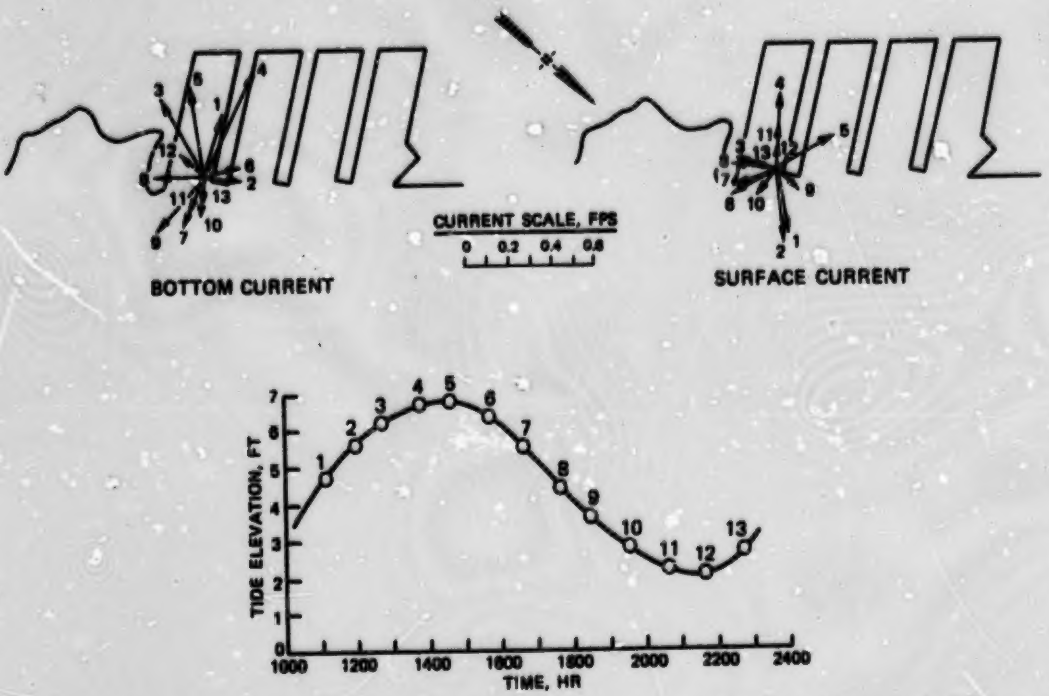

Figure A67. Surface and bottom currents, sta A5, Mare Island, 11 Sep 1976

slip on a rising tide and out of it on a falling tide. Surface flow directions are mixed. Maximum bottom velocity out of the slip was approximately 0.3 fps.

97. Figures A68 and A69 show variations of salinity and suspended sediment with depth and tide level at sta A1 and A5, respectively. Salinity in the main channel (sta A1) is minimum and nearly constant with depth at low water. Bracketing high water, a depth gradient exists with water in the lower part of the channel more saline than surface water by 2 to 3 ppt. In the slip entrance (sta A5), a similar trend exists, although the depth gradient at high tide is more gradual. Surface salinity changed little at sta A5 over a tidal cycle.

98. Near-bottom suspended sediments increased abruptly in the main channel (Figure A68) as flood currents reached their peak. A similar trend is suggested for ebb flow, although data were not taken during the maximum ebb current period. This phenomenon suggests resuspension of loosely consolidated bottom material by peak currents as a possible transport mechanism in the strait. Near-bottom suspended sediment in the slip entrance (Figure A69) is not as easy to correlate with observed currents (Figure A67). Average and maximum concentrations are relatively low. Abrupt increases occur as flow changes direction in the entrance (from in to out and vice versa), but the

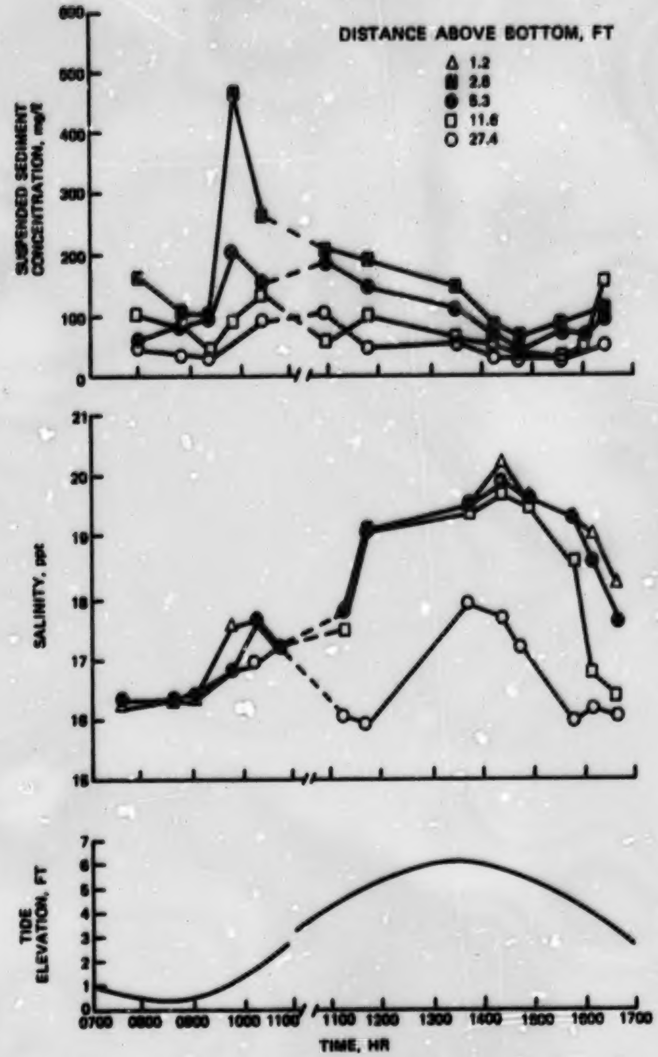

Figure A68. Suspended sediment, salinity and tide elevation, sta A1, Mare Island

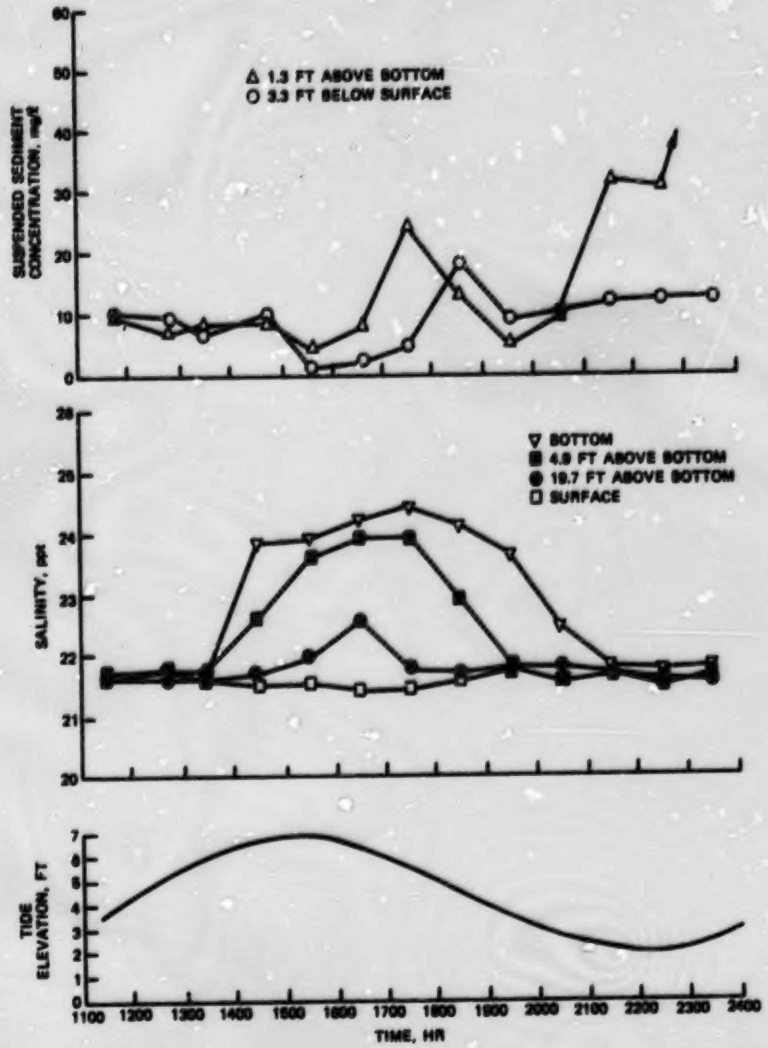

Figure A69. Suspended sediment, salinity, and tide elevation, sta A5, Mare Island

duration of these increases is short. The most significant increase coincided roughly with the change from outward to inward bottom flow, suggesting that slip sedimentation may be due in part to resuspended main channel sediments entering along the bottom on flood tide. Surface suspended sediments were uniformly low in the main channel and slip entrance.

99. Data at sta A1 were taken on 15 and 17 March 1977 for currents and 17 and 18 March for salinities and suspended sediments. In each case, complementary portions of a tidal cycle were monitored and later combined to produce the composites shown in Figures A66 and A68. Sta A5 data were taken synoptically over a complete tidal cycle on 11 September 1976. Freshwater discharges were low for all monitoring periods and tides were in the spring range, meaning that flow phenomena were tidally dominated.

100. In addition to Mare Island, SIO made field surveys of Navy facilities at Norfolk, Virginia, and Charleston, South Carolina. Results of these surveys were used to categorize the sedimentary environment of each site (Van Dorn et al. 1977):

> ...the three Navy facilities characterized by clay silt sedimentary environments...have much in common, and differ mainly in degree. All are saline estuaries within which, except during flood conditions, the normal tidal flux substantially exceeds the fresh water runoff. Each of these facilities happens to be situated within the tidal transition zone where fresh water runoff becomes admixed with ocean water, and whose length is of the order of the spring tide particle excursion (30-60 km).
>
> The transition zone is characterized by practically isotropic conditions at ebb water slack, and a vertical salinity gradient that increases during flood tide and decreases rather more abruptly during ebb. With relatively minor changes in physical and chemical properties,...clay minerals of micron size are brought down by fresh water in concentrations roughly proportional to runoff, and aggregate into flocculates of order 10 microns within the transition zone, where concentrations by weight are typically within the range 10-100 mg/liter. Here the still water sinking rate is of order 0.01 cm/sec, and the threshold stress required to resuspend newly settled floc is about one dyne/cm$^2$. In all three estuaries, the tidal bottom velocities in the main channels amply exceed this threshold...but Mare Island Strait is a backwater with no effective through flushing, and where settled floc tends to accumulate at nearly the same rate as in the adjacent basins.

> The facilities studied were in all cases finger pier basins, within which tidal velocities rarely exceed the floc suspension threshold (15 cm/sec), so that material brought in by any means tends to 'snow' down and remain. However, the observed floc concentrations are nowhere great enough to produce the observed accumulations of dense (300-360 gm/liter) bottom mud, so that some other mechanism must be invoked. ...the hypothesis that dense floc accumulations may be episodic, rather than prevailing, seems a more likely explanation for the principal differences in sedimentation rate between these three facilities.

101. Subsequent to initial laboratory tests of curtains, single jets, and jet arrays, further studies were made on individual nozzles. Three convergent nozzles of different diameter were tested in diatomaceous earth to determine their scour patterns as functions of nozzle diameter, discharge, and pressure drop. It was found that the power required to produce a bottom shear stress of $2.1 \times 10^{-3}$ lb/ft$^2$ at the scour pattern limits increased as the fourth power of maximum scour radius. In other words, doubling power supplied to the nozzle produced only a 19 percent increase in scour "reach," all other factors being equal. Using the laboratory results and nozzle discharge equation, a nomograph was constructed relating jet diameter, nozzle pressure drop, maximum scour radius, and power required to produce a bottom shear stress of $2.1 \times 10^{-3}$ lb/ft$^2$ at the scour pattern limits. This nomograph was used in sizing prototype jet arrays tested later in the study.

102. The first prototype agitation dredging test conducted by SIO involved monitoring the effects of propwash from a Navy tug on both newly dredged and highly shoaled bottoms. The tug used was a 2,000-hp single screw vessel which develops 53,000 lb of static thrust. The jet nomograph discussed above predicted that such a vessel should produce a maximum scour radius of 390 ft in newly deposited Mare Island sediment. Tests were conducted in a shoaled area near the inactive ship facility and a dredged area at Pier 23 (Figure A60). The tug operated in a stationary mode at each site for 10 min at slack high water during spring tide. Before and after bottom surveys showed no discernible scour as a result of these brief tests. The explanation given was that bottom sediments at both sites were not newly deposited, but instead were compacted with only a thin layer of unconsolidated sediment on top. Additionally, it was thought that test durations were too short to cause significant erosion in compacted material. Despite the lack of significant

erosion, it was noted that the turbidity plume generated by each test reached a maximum equilibrium distance of approximately 390 ft, providing some rough correlation with jet performance predicted by the nomograph.

103. A prototype scouring jet array was installed in the docking slip south of Pier 23 in February 1977 (Figure A70). The array, shown in Figure A71, covered a 10,880-$ft^2$ area with 70 jet nozzles of 0.8 in. in diameter. Each jet had a design discharge of 114 gpm. The area in which it was installed had to be dredged to project depth (33 ft below mllw) to a tolerance of ±0.5 ft. A similar area was prepared in the slip south of Pier 22 as a control for determining sedimentation prevented by the array.

104. The jet array was driven by a 150-hp centrifugal pump supplying 1,600 gpm at 100 psi. An automatic control system started the pump 4 hr after low tide. Water was supplied to each pair of pipe branches in the array for 7 min, for a total jetting time per tidal cycle of 35 min. The pipe branch pairs were operated sequentially toward the strait by automatically controlled pinch valves on each branch. This operating cycle was determined based on 2-week measurements of a near-bottom current meter placed slightly straitward of the slip entrance. The meter showed sustained ebb flow in the strait averaging 1.3 fps beginning 1 to 2 hr before high tide. Currents during the rest of a tidal cycle were weak and variable.

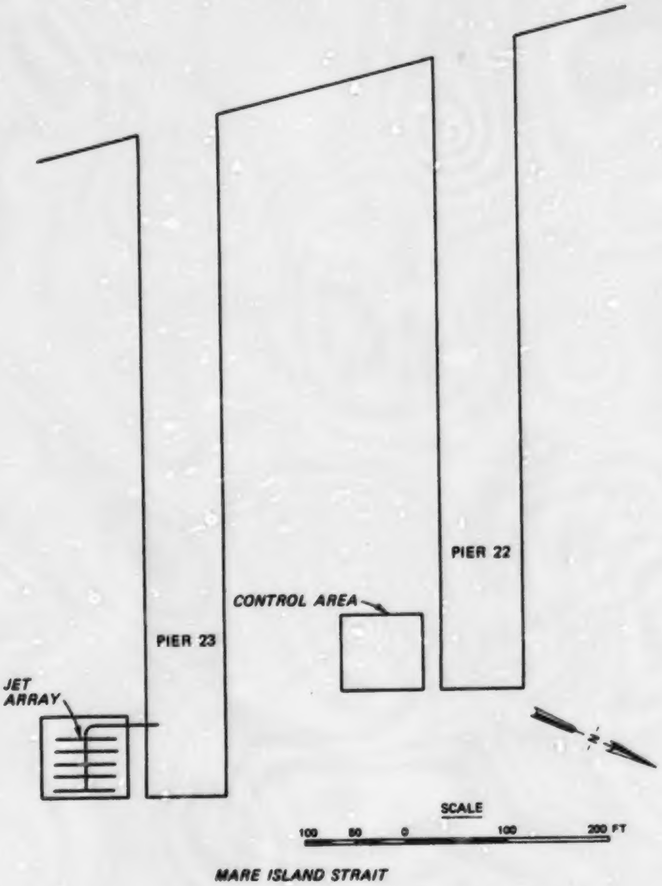

Figure A70. Jet array and control area, Piers 22 and 23, Mare Island

105. The jet array operated automatically for approximately 4 months (March through June 1977). Problems with the pinch valves, burial

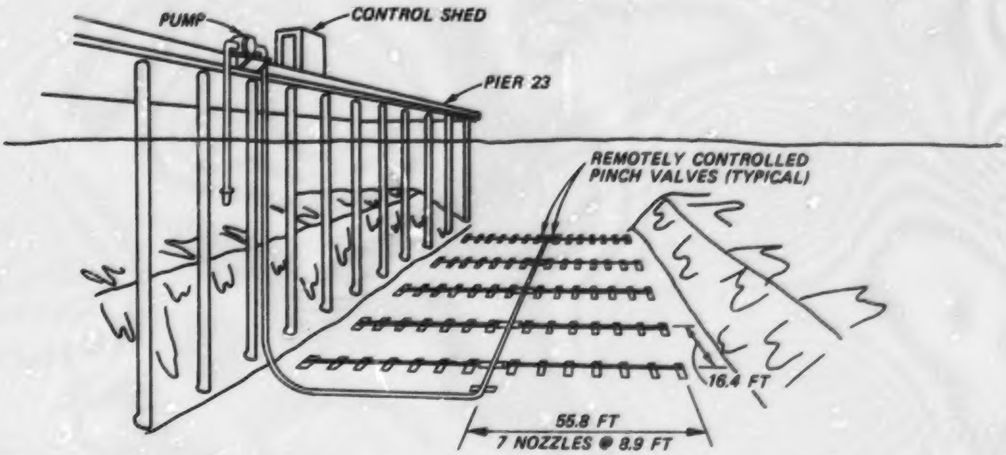

Figure A71. Jet array, Pier 23, Mare Island

of part of the array by material slumping from the undredged bottom, and lack of fresh sediments in Mare Island Strait caused the array to be shut down in July 1977. Effectiveness of the array in preventing sedimentation is summarized in Figure A72, which shows average bottom elevation changes versus time for three survey tracks in the control area, area in the docking slip behind the jet array, and area over the array. Shoaling in the control area proceeded at a gradually decreasing rate until mid-May, at which time it ceased until July. The area over the array remained essentially free of shoaling for the entire monitoring period, while the area behind the array accumulated approximately half the sediment which settled in the control area. These results were interpreted as showing that the jet array was completely successful in preventing shoaling over it and partially successful in blocking sediment intrusion into the docking slip. Sediment intrusion was also inhibited by the fact that the array was located in a specially dredged area with a 6.5-ft sill at the rear. Since suspended sediment was confined mostly to the lower 3 ft of the water column during the monitoring period, the sill formed a barrier to sediment intrusion.

106. The next jet array tested is shown in Figure A73. This array consisted of 25 jets spaced 8.8 ft apart along a 212-ft pipe. The pipe was installed at Berth 6 (Figure A74) 20 ft below mllw with the jets angled downward 25 deg below horizontal. The purpose of this array was to maintain a submarine berthing space at 34-ft depth, with or without a submarine present, the space having been dredged prior to array installation.

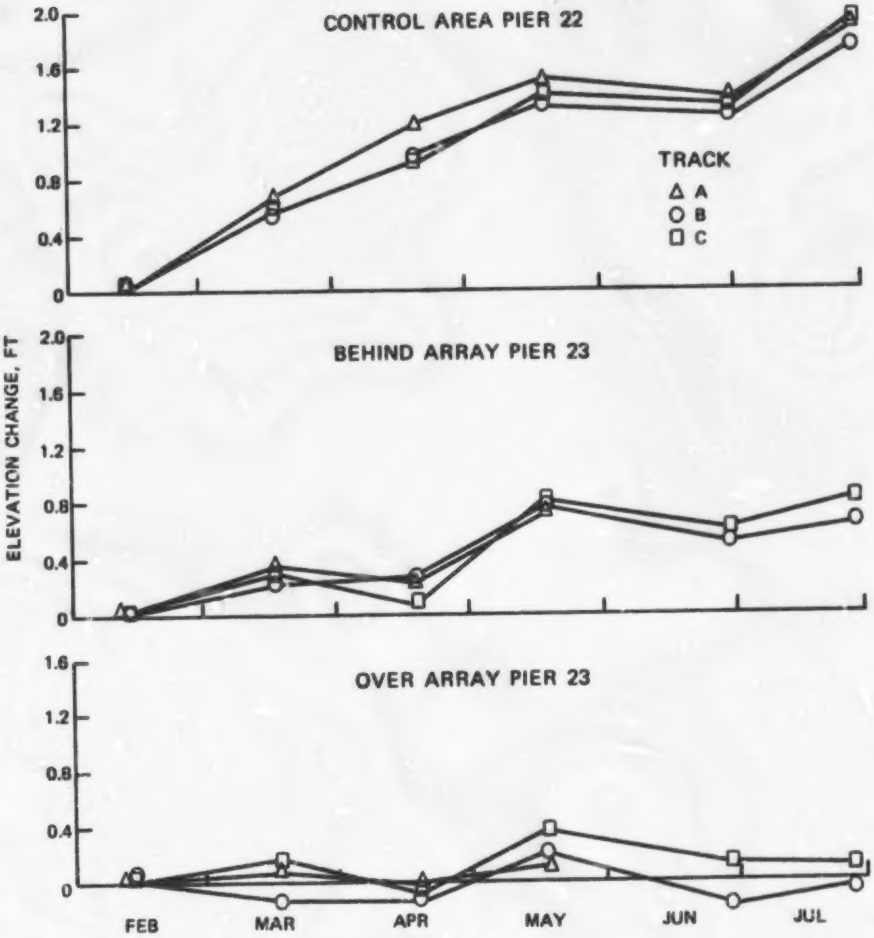

Figure A72. Bottom elevation changes, jet array and control areas, Mare Island

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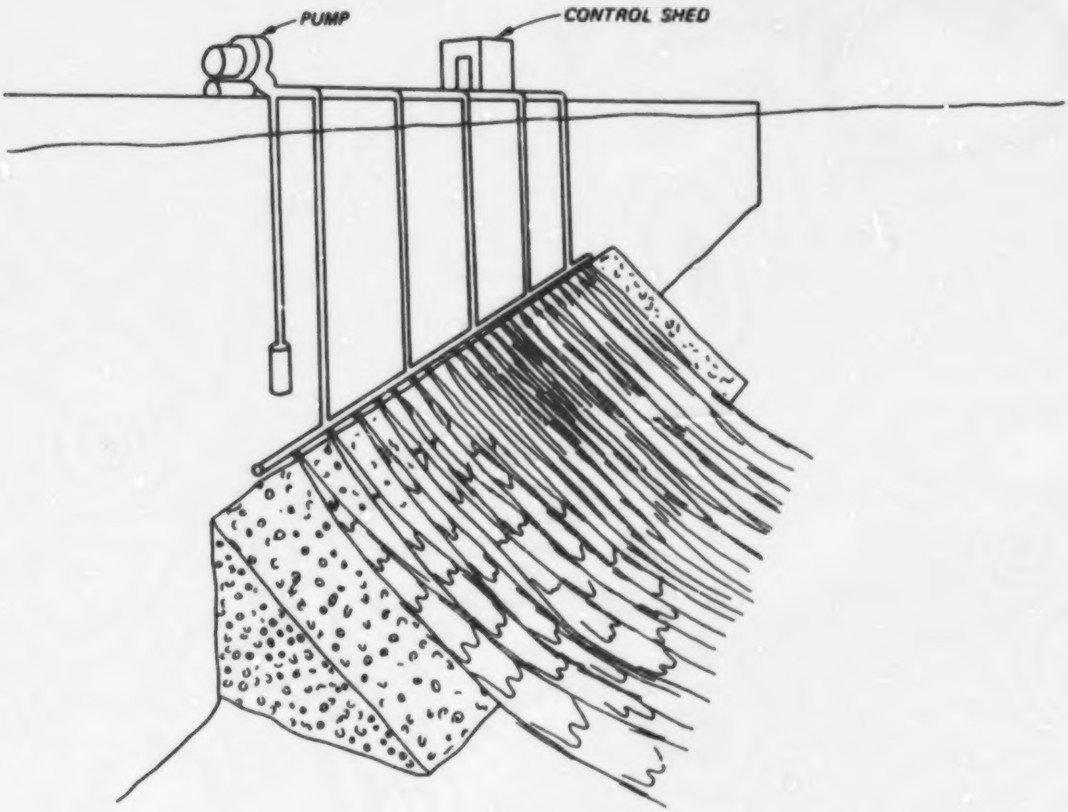

PUMP
CONTROL SHED

Figure A73. Jet array, Berth 6, Mare Island

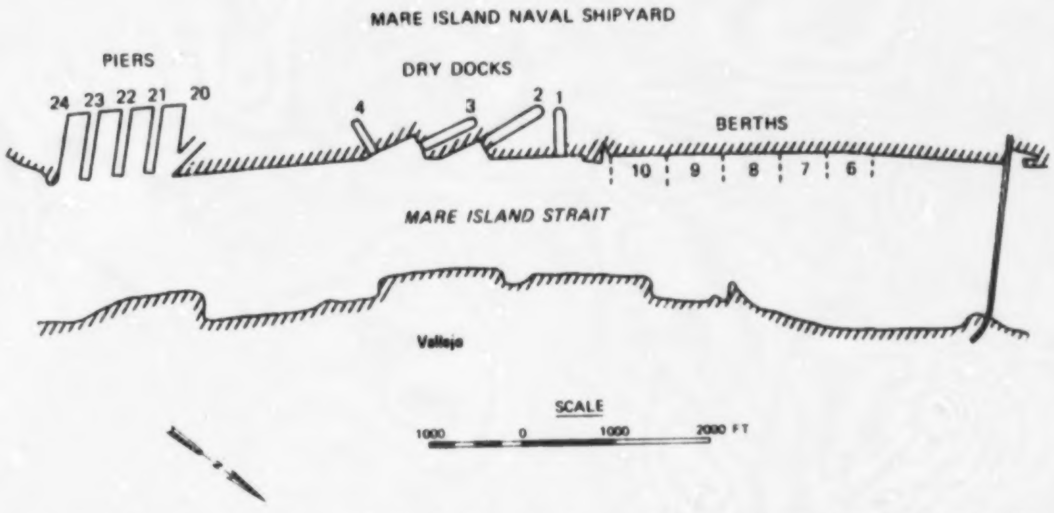

MARE ISLAND NAVAL SHIPYARD
PIERS
24
23
22
21
20
DRY DOCKS
4
3
2
1
BERTHS
10
9
8
7
6
MARE ISLAND STRAIT
Vallejo
SCALE
1000
0
1000
2000 FT

Figure A74. Berths, drydocks, and piers, Mare Island Naval Shipyard

107. The array jets were operated in groups of five by an automatic control system; each jet discharged 380 gpm. A 150-hp pump supplied 1,900 gpm of water at 90 psi to the jet groups, which were operated sequentially from north to south. For the first 2 months of testing (February-April 1978), the jet array operated a total of 1 hr per tide cycle. Bottom surveys showed that jet scouring was not extending far enough, so the operating time was increased to 4 hr per tide cycle. Operations continued until June 1978, when valve failures caused the experiment to be terminated.

108. Figure A75 summarizes bottom conditions at Berth 6 during jet array operations by showing average bottom depth versus time along lines parallel to the berth wall at distances of 20, 30, 40, and 50 ft away. The effect of increasing operating time from 1 to 4 hr per tidal cycle is the phenomenon most evident in this figure. When the array was operated 1 hr per tidal cycle, bottom depths beyond 20 ft from the berth wall changed in an

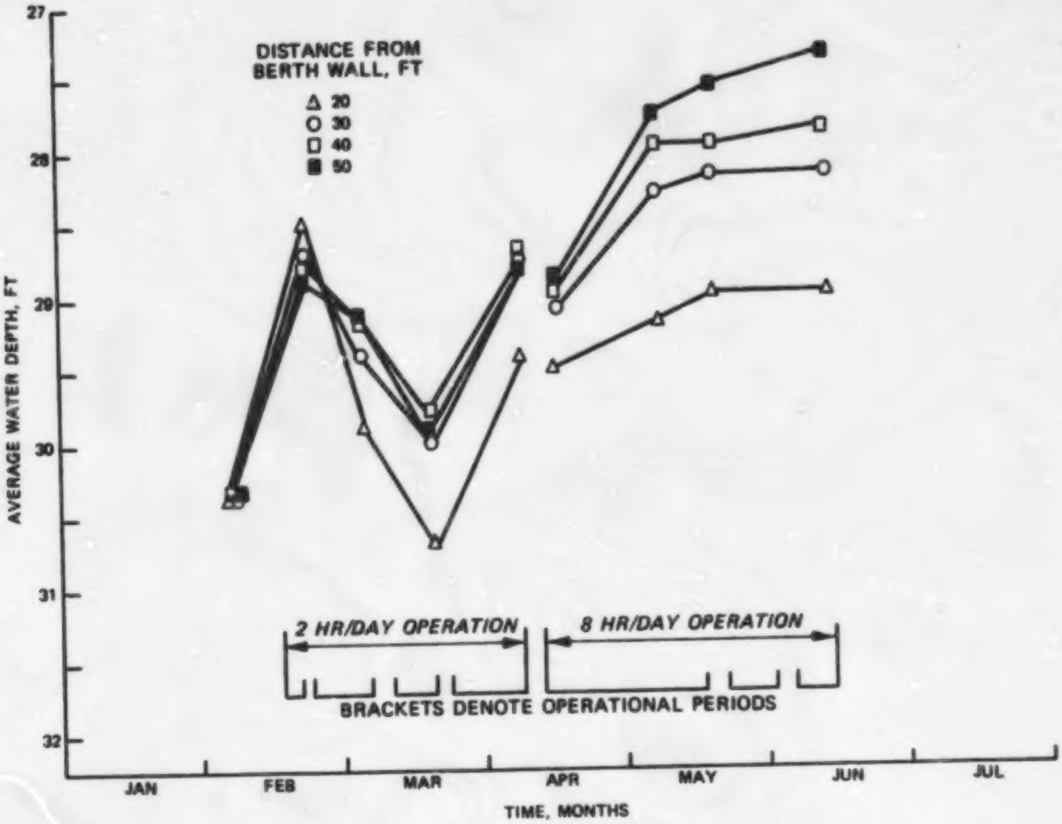


Figure A75. Water depth versus time, Berth 6, Mare Island

identical fashion. Increasing the operating time caused depths to vary according to distance from the wall, showing that jet influence was extended to at least 50 ft from the wall. Even with increased operating time, however, the jet array could not prevent shoaling completely in the submarine berth. Water depths at the experiment conclusion were 0.5 ft less at 20 ft, 0.8 ft less at 30 ft, 1.2 ft less at 40 ft, and 1.6 ft less at 50 ft than at the beginning of the 4 hr per tidal cycle operating period, which spanned 68 days with two brief interruptions. It was concluded that bottom stress levels of 8.3 to $10.4 \times 10^{-3}$ $lb/ft^2$ needed to be sustained for a longer fraction of each tidal cycle for such an array to be effective in high deposition rate areas.

109. With these results in mind, additional laboratory experiments were conducted in 1979 to measure the bottom shear stress of a single jet. These experiments showed that bottom shear stress decayed more rapidly with distance from the jet than had been assumed previously. Using this new information, the submarine berth jet array was redesigned so the jets would generate a theoretical bottom stress of $9.6 \times 10^{-3}$ $lb/ft^2$ at 50 ft from the berth wall. The total number of jets was reduced to 10, spaced 22.8 ft apart. Discharge of each jet was increased to 1,910 gpm, and the automatic control system reworked to operate one jet at a time. The system required a centrifugal pump producing 1,910 gpm at 92 psi, driven by a 140-hp motor.

110. This new array was installed in May 1979 at Berth 7 (Figure A74) and was still operating almost 2 years later, although numerous interruptions had occurred. These interruptions made it difficult to assess long-term effects of the jet array on shoaling. However, three bathymetric surveys made in 1979 indicate possible short-term results, as summarized in Figure A76. This figure shows bottom depths versus distance from the berth wall at two different jets and a control survey line in the Berth 8 area, each for three different dates. Jet 3 had a deflection angle of 29 deg downward from horizontal, while jet 8 deflected 35 deg. The 30 March survey was taken immediately after dredging Berth 7 to prepare for array operation. However, the array did not begin operation until 30 May, so the 17 May survey reflects unhindered sediment accumulations at all locations. The 9 August survey shows the effects of 70 calendar days of array operation, 4 hr per tidal cycle. During these 70 calendar days, the system was nonoperational for approximately 22 days due to various malfunctions.

111. The main short-term effect visible in Figure A76 is that jet 8

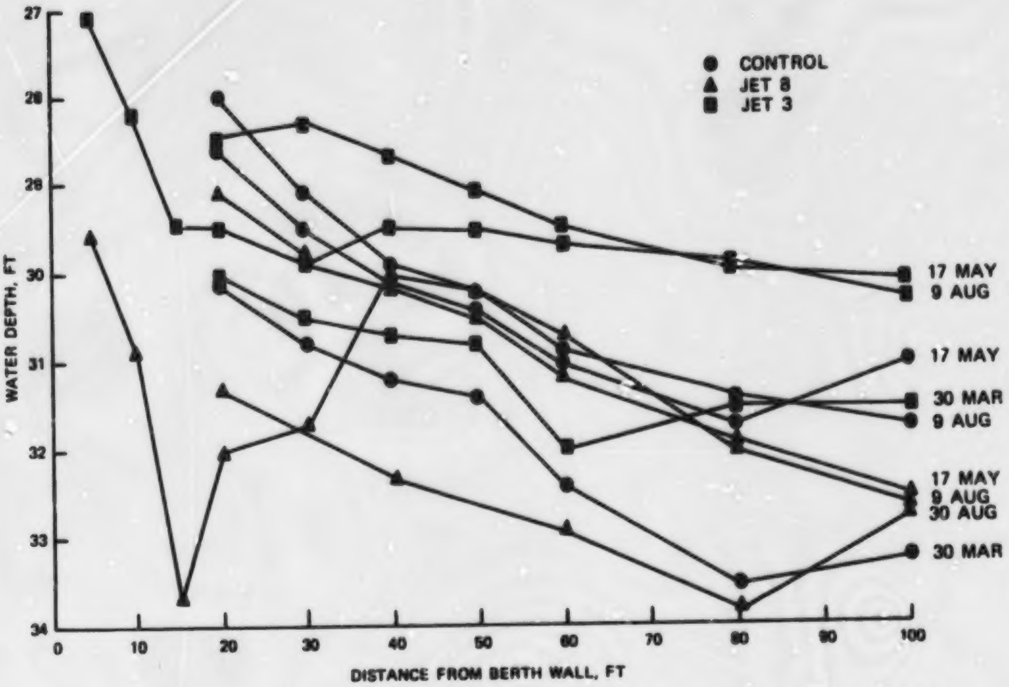


Figure A76. Water depths, Berth 7 jet array, Mare Island

created a scour hole extending to about 35 ft from the berth wall, and that apparently some of this scoured material was deposited beyond the hole from 35 to 70 ft out. Jet 3's effects were much more moderate, but still extended only to 40 ft out, relative to changes occurring along the control line. The difference in these two effects was attributed to the deflection angle of jet 8 being 6 deg greater than jet 3. Both jets removed some deposited material and, presumably, hindered deposition within their area of influence. However, the array began operation with bottom elevations several feet higher than intended, so it is difficult to judge from the measured effects how the jets would have performed under design conditions. The 9 August bottom elevations at both jets were higher than the 30 March postdredging elevations except in the deepest portion of the jet 8 scour hole, meaning that the jets were only partly effective in reestablishing design bottom conditions. Whether this was due to an inability to excavate consolidated material or whether deposition during nonoperational periods replaced previously excavated material is not known.

112. Another agitation dredging device tested briefly by SIO at Mare Island was the Ventra-Vac system. Tests of this system and a simular one called Helixor at Grays Harbor, Washington, are described earlier in this

appendix. The Ventra-Vac installation at MINSY consisted of two units placed on either side of the entrance to Dry Dock 2 (see Figure A74 for dry dock locations). Figure A77 shows an artist's concept of the installation and a Ventra-Vac unit. The south unit was located with the top 10 ft below mllw, while the north unit had to be placed 4.5 ft higher due to concrete rubble and a mud bank on the bottom. Each unit received 110 to 120 scfm of air and produced significant boils of air and water at the surface. Sediment transport at the surface was visible only as a small amount of sediment clinging to air bubbles.

113. The Ventra-Vac units operated continuously from 12 December 1979 to early February 1980. During this period, three bathymetric surveys were made along a line connecting the two units, the results of which are shown in Figure A78. In the 27 days between 12 December and 8 January, approximately 3 ft of sediment accumulated uniformly between the units, while little additional shoaling occurred in the 22 days between 8 and 20 January (this apparent phenomenon of episodic shoaling will be discussed in succeeding paragraphs). Since these shoaling rates corresponded with ones measured at other MINSY locations at the same time, it was concluded that the Ventra-Vac units had no noticeable effect on bottom elevations except for a scour hole 6 to 8 ft in diameter observed by divers at the base of each unit.

114. Two types of environmental phenomena were cited during the SIO work at Mare Island as having had direct effects on the experimental regime. The first of these was an extreme variability in rainfall, hence runoff and stream sediment load, from year to year during the study. The winters of 1975/76 and 1976/77 were the driest in two decades along the Pacific coast, causing flows in the San Joaquin and Sacramento Rivers to be reduced to minimal levels. Riverine sediment influx to the study area was negligible except for a single episode in January 1977, when a storm raised river discharge to normal levels and produced a sediment "pulse" which decayed over several months. Looking at Figure A72, the effects of this pulse appear as a gradually decreasing shoaling rate in the control area from February to mid-May 1977, at which time it had decayed completely and shoaling had ceased. The abrupt increase in July was attributed to dredging in an adjacent area, not to sediment entering Mare Island Strait.

115. Following this 2-year drought, the winter of 1977/78 was the wettest in two decades, producing a completely different sedimentary environment.

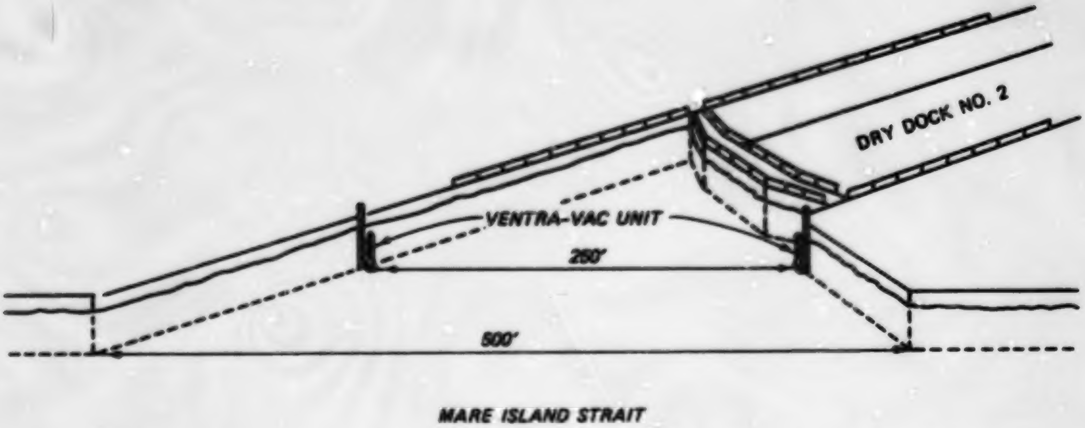

DRY DOCK NO. 2
VENTRA-VAC UNIT
250'
500'
MARE ISLAND STRAIT

Figure A77. Ventra-Vac installation, Dry Dock 2, Mare Island

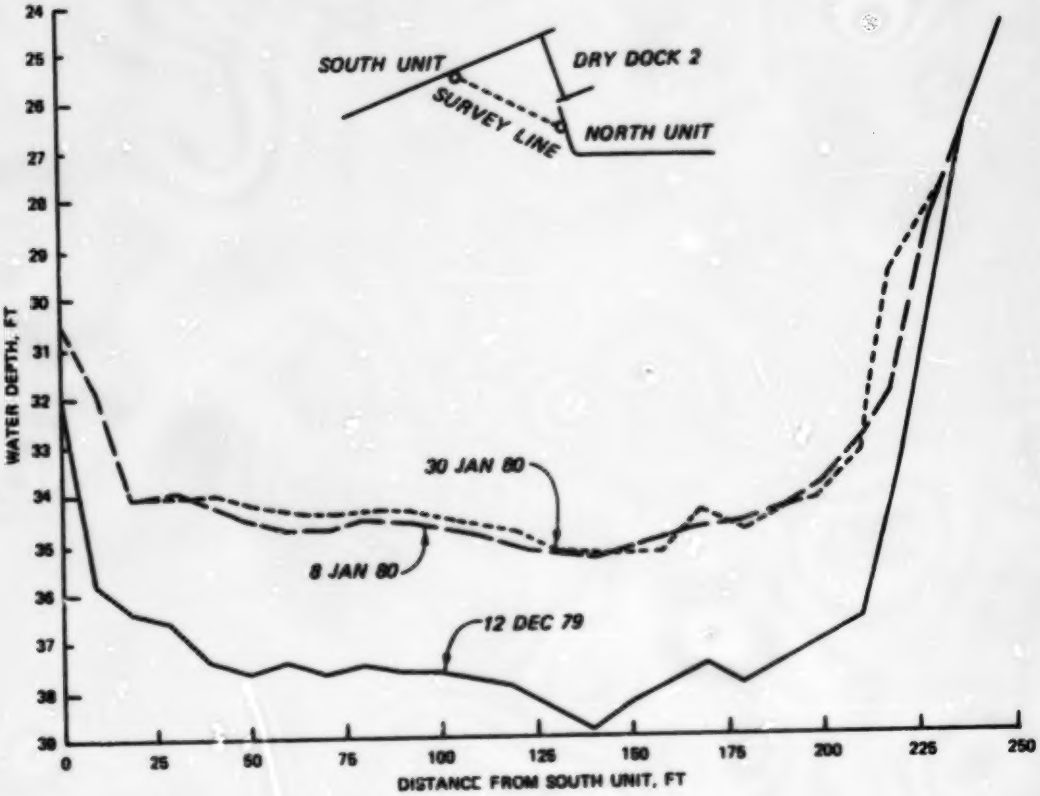


Figure A78. Water depths between Ventra-Vac units, Dry Dock 2, Mare Island

118

For example, shoaling at one control station in the test area was 2.5 ft in 1977 and 13.1 ft in 1978. This anomalous season was followed by a winter of average conditions in 1978/79, while the 1979/80 winter was again one of exceptionally high rainfall. This extreme variability over the test and monitoring program was fortunate in that data on the sedimentary environment were collected over a wide range of conditions. However, it was unfortunate that the various systems tested had to operate in such different regimes, making it more difficult to compare their performance.

116. The other phenomenon cited as having affected experimental conditions was the periodic occurrence of rapid sedimentation events, termed "mud storms," which could deposit several feet of shoaling in a few days. Such events usually occurred during the final stages of storm runoff when rivers were still carrying a high concentration of suspended sediment, but freshwater flow had decreased enough that salinity in the strait reached a flocculation threshold. Six such events were documented by echosounder between 12 December 1979 and 4 June 1980. Each event deposited 2 ft or more of fluid mud in a short period of time, for a total accumulation of 14 ft in 6 months. Obviously, such a phenomenon could have tremendous effects on the design, operation, and success of a fixed agitation dredging system.

117. As an adjunct to the jet arrays tested at Mare Island, Bailard (1980) discussed a procedure for selecting a jet array design with minimum annual and capital costs. Assuming a certain required bottom shear stress, jet duty cycle, water velocity in pipes, energy inflation rate, rate of return, and system life, he presents graphs for arriving at the optimal system design. An interesting result of one example in his analysis is that the system Savings Investment Ratio (SIR) relative to dredging costs did not reach 1.0 until dredging costs exceeded $6.00/sq yd of bottom. An SIR of less than 1.0 indicates the investment is economically unattractive, although such figures are obviously somewhat subjective.

## Propeller Jet Erosion

118. Slotta and Higgins (1976) describe an analytical and model study into the hydrodynamic and scouring characteristics of deflected propwash. They present equations and experimental results from other investigators on the hydrodynamic characteristics of submerged jets impinging on flat

boundaries. They point out that one investigator has shown that jets whose velocities pulsate have greater eroding potential than steady-state jets. The pulsating condition is analogous to the flow field of a propeller wake jet. They discuss some characteristics of scour and scour hole patterns and relate them qualitatively to jet hydrodynamics. This is followed by a presentation of criteria used to scale hydrodynamics and sediment transport for constructing scale models of a propwash agitation craft.

119. Two models of the vessel *Sandwick*, discussed previously in this appendix, were built by Slotta and Higgins to scales of 1:12 and 1:40. Propeller dimensions and speed were also scaled. Sediment used in the scour tests was considerably larger than that needed for similitude; however, the investigators felt that the geometric, flow field, and Reynolds' similitudes attained would produce scour patterns analogous to prototype.

120. Using these models, the three-dimensional velocity distribution behind the *Sandwick* was determined for various deflector door angles and found to correspond reasonably well with work by other investigators. Scour pattern planforms were found for various combinations of water depth and propeller usage. Depth and volume scour was studied for different deflector door angles and water depths. Maximum water depth for feasible propwash agitation was estimated to be four times the agitating vessel's draft. The time required for a scour pattern to reach equilibrium configuration was determined to be 600 times the ratio of sediment fall velocity to wake velocity at the propeller. Large-scale wake turbulence was observed to be a major factor in propwash agitation, producing the pulsating effect mentioned earlier. Criteria and means for regulating and optimizing these pulsations were areas suggested for further research.

121. A significant amount of research has been conducted, mostly in Europe, on the mechanism and prevention of bottom and side slope erosion by propeller jets (propeller wash) from vessels. Bergh and Cederwall (1981) reported $1.1 million damage to harbor structures in Sweden over a 10-year period as a result of such erosion. In the upper Rhine River, bottom agitation by propeller jets of moving vessels at low water stages causes bed material transport in excess of that which would occur by natural currents (Felkel 1977), contributing to an undesirable lowering of the riverbed. Ballin (1977) describes model studies of localized severe bottom scour caused by grounded ships freeing themselves and barge tugs getting under way from a dead stop.

He also discusses sediment ridges formed by the scoured material which can be 4 ft or higher.

122. Although such erosion is presented in these references as a problem, its mechanism has obvious similarities to the propeller wash agitation dredging described earlier in this report in the Chinook Channel, Missouri River, Pacific Northwest, Tillamook Bay, Savannah Harbor, and Mare Island. Bergh and Cederwall (1981) present graphs of bottom velocities and formulas for estimating the risk of bottom erosion from a stationary vessel and for calculating the size of bed armor required to prevent such erosion. Ballin (1977) shows scour patterns, maximum scour depths, and maximum ridge heights caused by vessels of different power outputs and propeller speeds getting under way from a dead stop. The Delft Hydraulics Laboratory (1979) is engaged in a comprehensive study of ship effects on bank and bottom erosion, a portion of which is the investigation of propeller jet erosion. Blaauw and Van de Kaa (1978) describe results of the first part of this investigation, which include a method for determining jet velocity distribution and bottom erosion caused by the jet. Other research in this area can also be cited. In summary, it appears that the theoretical and experimental basis has been established, at least in part, for a fundamental study of propeller wash agitation dredging.

## People's Republic of China

123. Luo and Gu (1981) describe agitation dredging in fine-grained material in China, usually in conjunction with the release of water through tide gates. A test was conducted from July to August 1967 at Xinyang Estuary Gate, where two boats dragged a rake over the bottom at the same time water was released in an ebb direction. The result of this agitation was a threefold increase in material removed over that accomplished by water release alone. Laboratory experiments were also conducted on this method, and showed that rakes have optimum dragging speeds (Figure A79), that the optimum time to rake varied with location and tide, and that different rake forms were needed for different soil types.

124. Rake dragging has also been used at several locations in Heipei Province to increase sediment transport by ebb currents downstream of tidal barriers. In the Dou River Estuary, raking operations over a 36-day period increased channel cross sections to 80 percent of their design area at a cost

estimated at 10 percent of dredging to achieve the same results.

125. An improvement on the basic rake, called an aerated rake, was designed and tested in the laboratory and field in the late 1970's. A conventional rake of the type used at other locations was modified to incorporate a series of holes on the back through which compressed air could be pumped. The rising air generated vertical turbulence and currents to enhance bottom agitation and distribute agitated sediment more uniformly throughout the water column. A field test conducted in 1977 at the Yong River Estuary mouth used a 13-ft-wide rake with air supplied at the rate of 320 cfm. By measuring suspended sediment concentrations, in situ sediment properties, and bottom changes, it was determined that aeration increased average suspended sediment concentrations by 1.15 to 2.02 times that obtained with a conventional rake, and that the remaining bottom sediment was of greater density. Similar tests in 1979 in the Xinyang Estuary below a tidal gate showed an increase in average suspended sediment concentration of 2.8 times due to aeration, with concentrations in the upper water layer being 4.4 times as great. Aeration reduced the cost of removing a unit volume of bottom material to one-third that of a conventional rake.

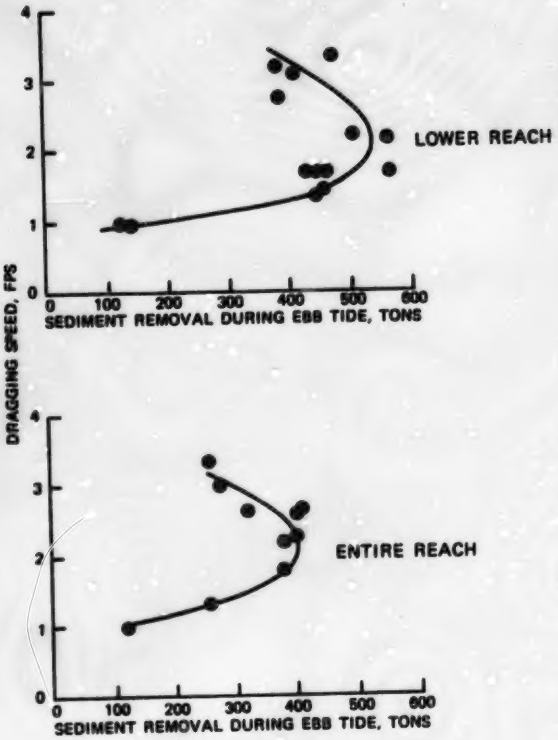


Figure A79. Sediment removal versus rake dragging speed

126. Luo and Gu also note continuing experiments with aerated rakes and water jets attached to rakes. They summarize China's experience with rake agitation dredging as follows:

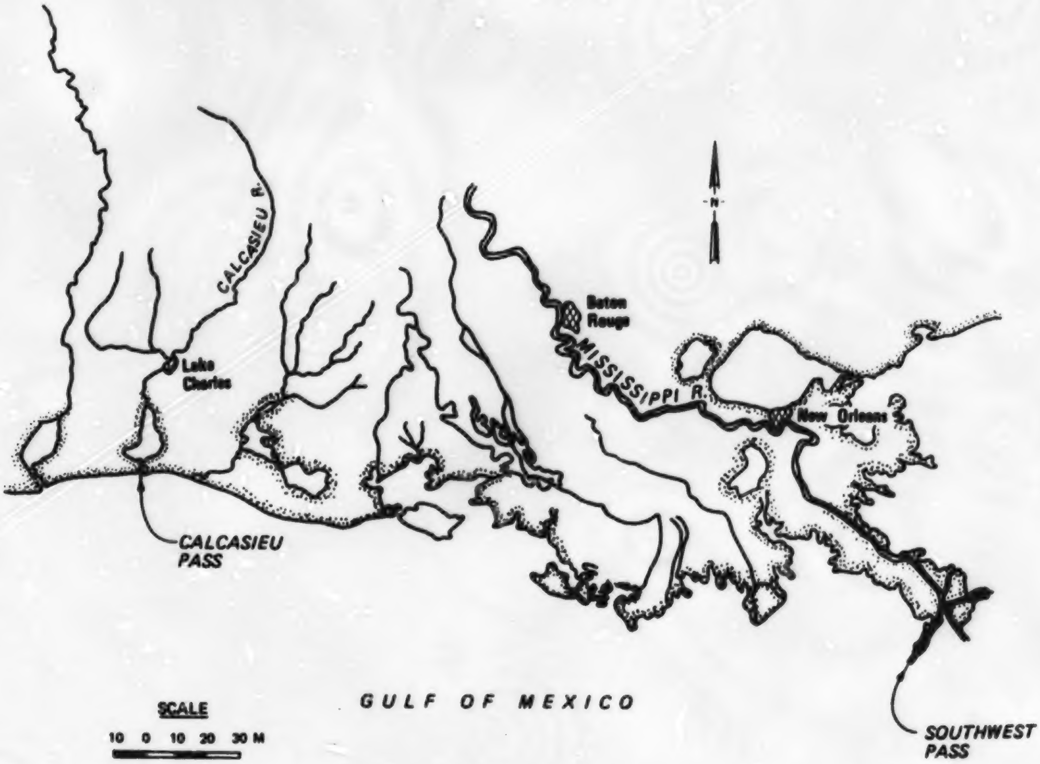
a. Best results were obtained in material with median grain size less than 0.1 mm. In one case, agitated material of 0.005-mm-median grain size was carried out of the estuary on one ebb tide. They note, however, that the fine-grained material can be difficult to agitate when it is compacted.

b. The aerated rake was more efficient in water depths greater than 10 ft, and could be used even on a bar at the estuary mouth.

c. The Chinese maintain river channels below tidal gates mainly to provide discharge routes for floodwater during the rainy season. Therefore, the optimum time to dredge these channels, whether by agitation or other means, is difficult to predict. If dredged too far in advance of flood periods, the channels will refill with material entering from the sea or downstream areas.

## Louisiana Gulf Coast

127. Agitation dredging by hopper dredge has been a primary means of navigation channel maintenance for a number of years at two locations on the Louisiana Gulf Coast--Southwest Pass and the Calcasieu River approach channel (Figure A80). This dredging is performed in the same manner as described earlier for the Delaware Estuary, in which the dredge raises material into its hoppers and allows it to spill overboard at the water surface. The two operations, although often performed by the same equipment, are different enough to be treated separately.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Lake Charles
Baton Rouge
MISSISSIPPI R.
New Orleans
N
CALCASIEU PASS
GULF OF MEXICO
SCALE
10 0 10 20 30 M
SOUTHWEST PASS

Figure A80. Location, Southwest Pass and Calcasieu River

128. Southwest Pass is one of three major outlets for the Mississippi River through its delta to the Gulf of Mexico (Figure A81), the other two being South Pass and Pass a Loutre. Southwest Pass is the main navigation channel between the Gulf of Mexico and Head of Passes and the main passageway for Mississippi River traffic entering and leaving the gulf. It carries approximately 40 percent of Mississippi River discharge reaching the Head of Passes (US Army Engineer District, New Orleans 1973). The range of discharge in Southwest Pass over a 35-year period of record was 12,200 to 460,000 cfs. The entire pass is tidal, with flow varying as much as 300 percent in one tidal

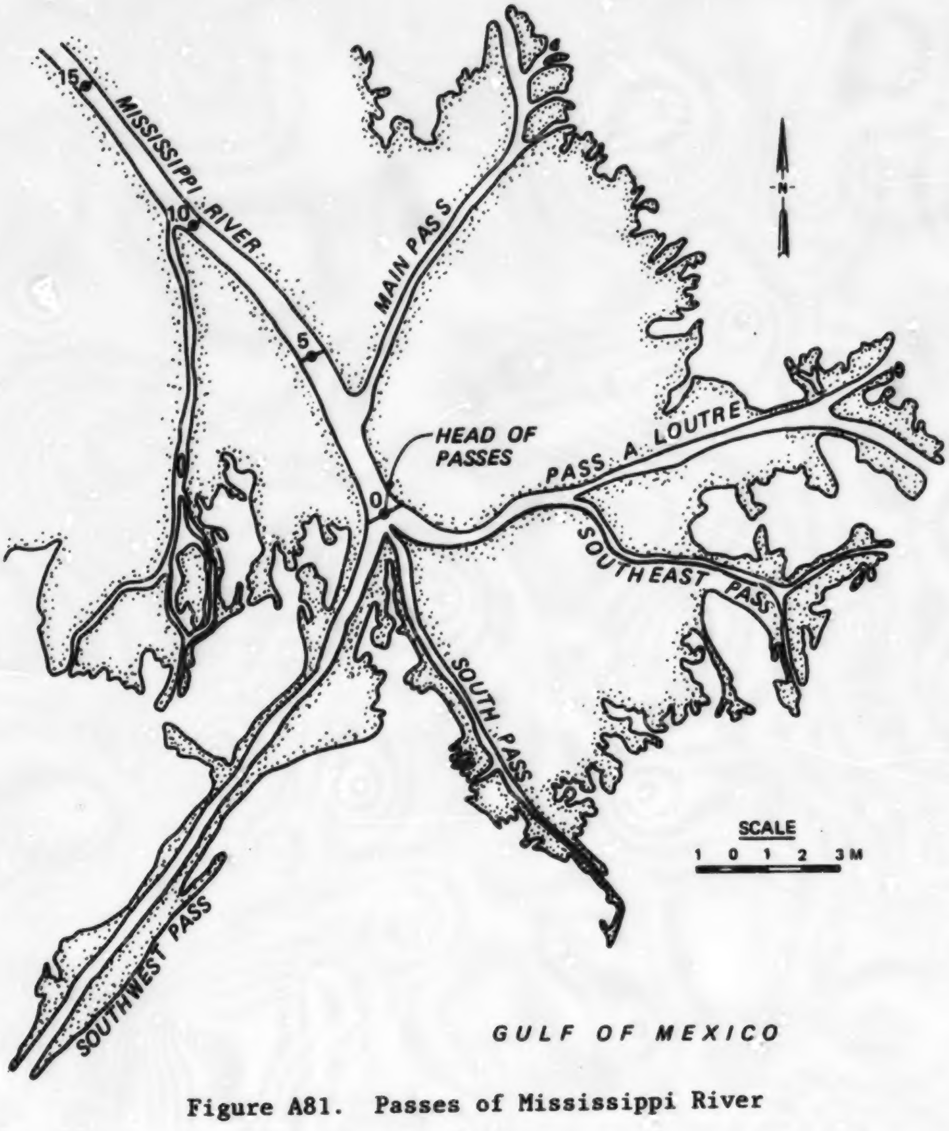

15
MISSISSIPPI RIVER
10
5
MAIN PASS
N
HEAD OF PASSES
0
PASS A LOUTRE
SOUTHEAST PASS
SOUTH PASS
SCALE
1 0 1 2 3M
SOUTHWEST PASS
GULF OF MEXICO

Figure A81. Passes of Mississippi River

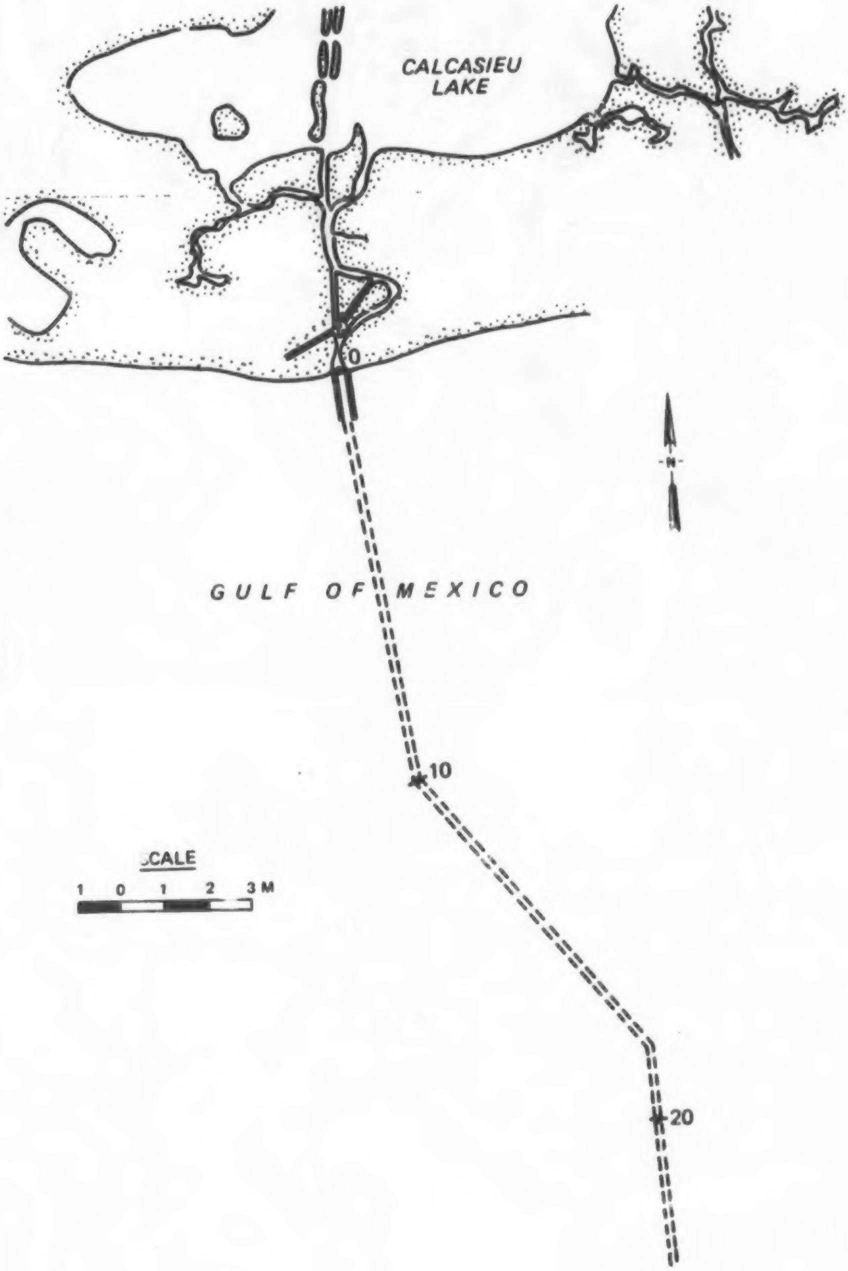
122

cycle. Mean tide range at the mouth of Southwest Pass is 1.5 ft, and spring tide is 2.0 ft. Saline water intrudes into Southwest Pass as a well-defined wedge, the extent of intrusion depending mainly on the amount of freshwater discharge. At lower total river discharges (<100,000 cfs), the wedge tip extends to a shallow river crossing above New Orleans (Simmons and Rhodes 1965). Above 200,000-cfs total river discharge, the tip reaches only to Head of Passes or below. At high discharge, the saltwater wedge remains in the lower part of Southwest Pass, causing accelerated shoaling which is removed by agitation dredging. During these periods, there is continuous gulfward flow in the upper water layers with surface velocities as high as 10 fps. In the lower water column downstream of the saline wedge tip, net transport is upstream.

129. Sediment load in the Mississippi River below Baton Rouge averages 950,000 tons per day, with approximately 70 percent of this carried as suspended load. The average suspended load is 7 percent fine sand, 38 percent silt, and 55 percent clay. Bedload composition is more variable but rarely includes material coarser than fine sand. This bedload is stopped abruptly when it encounters the saline wedge tip, causing concentrated shoaling. Suspended material is carried out into the gulf by the overlying fresh water, settles out as the freshwater jet diffuses, and is then transported back into Southwest Pass by net upstream movement in the lower water column.

130. The situation in the Calcasieu River approach channel (Figure A82) is considerably different from that in Southwest Pass. The channel extends approximately 23 miles from the jetties out to the 42-ft contour in the Gulf of Mexico. Surface currents change direction with the tide and are generally weaker than those in Southwest Pass. For instance, a field survey made in the late 1940's when the channel was 30 ft deep instead of the present 42 ft showed maximum surface ebb velocities of 5 fps and flood velocities of 3.4 fps in Calcasieu Pass, several miles upstream of the jetties (USAEWES 1950). A 3-day survey conducted in 1976 in the approach channel 4 to 7 miles out from the jetties (Thibodeaux and Grimwood 1978) measured surface velocities of 1 to 1.8 fps. Although the direction was not given, it is believed to be transverse to the channel, based on other statements made. There is a predominant littoral current from east to west across the approach channel which is a major factor in the agitation dredging operation (Kennedy 1963).

131. The sources of shoaling in the Calcasieu River approach channel appear different in nature from those for Southwest Pass. The lower river


CALCASIEU
LAKE
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GULF OF MEXICO
10
SCALE
1 0 1 2 3 M
20

Figure A82. Calcasieu River approach channel

12 b

carries little suspended load and virtually no bed load. Bank erosion and contributions from lakes next to the channel may be a major source of suspended sediment in the lower river, but how much of this deposits in the approach channel is not known. Certainly, the approach channel functions as a trap for bottom sediment moved by littoral currents, and this may be a primary source of channel shoaling. Material in the hopper of a dredge operating in miles 4 to 7 of the approach channel had a median grain size of 0.0015 to 0.05 mm. Bottom material was probably finer, since the hopper load resulted from some degree of hydraulic sorting in the dredging and overflow progress.

132. Agitation dredging has been performed in Southwest Pass for over 50 years, primarily since 1947 by the Corps of Engineers hopper dredge *Langfitt* (Figure A83). The *Langfitt* is a 3,000-cu-yd-capacity hopper dredge 352 ft long, with two side-mounted drag arms and a maximum dredging depth of 62 ft (Roorda and Vertregt 1963). She has a speed of 12.8 knots loaded and 15.3 knots light, and is powered by two propellers, each with 3,000 hp. The two dredge pumps are each driven by a 1,150-hp electric motor. The *Langfitt* is equipped to discharge hopper overflow conventionally, via overflow troughs and discharge chutes. However, when operating as an agitation dredge, it discharges through emergency gates located much lower in the hopper sides (Figure A84). This reduces the residual load of coarser material carried in the

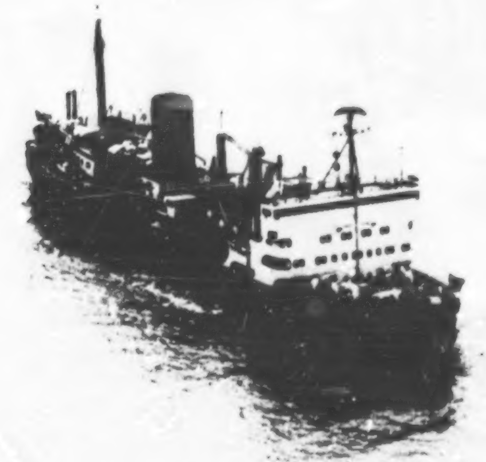

Figure A83. US Army Corps of Engineers hopper dredge *Langfitt*

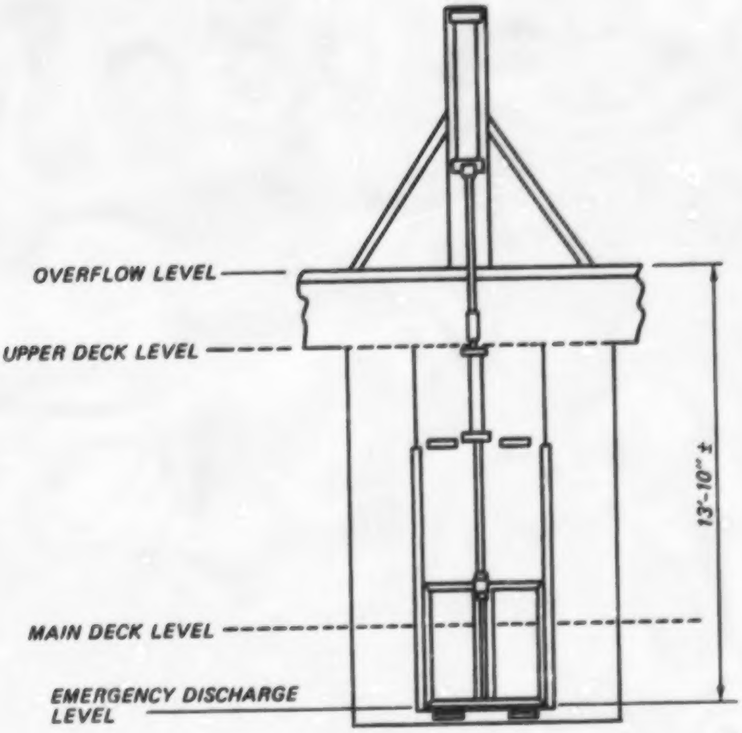

Figure A84. Typical emergency discharge gate, hopper dredge *Langfitt*

hoppers to approximately 1,300 cu yd, allowing better maneuverability and decreased draft (Scheffauer 1954). Also, material is discharged near the water surface instead of being dropped from a greater height, which would force it down into the water column as a denser jet and reduce its suspended travel time.

133. The *Langfitt* operates mostly by agitation, and is the primary means of maintaining Southwest Pass from mile 18.8 to outside of the jetty tips. Agitation dredging is performed during higher river stages, beginning as early as January and ending as late as July. During unusually high discharge, such as occurred in 1973-1974, other Corps hopper dredges may be used to augment the *Langfitt*. Figure A85 shows three hopper dredges working in Southwest Pass in 1973 and tugs trying to free a grounded vessel in the foreground.

134. Scheffauer (1954) gives a detailed account of dredging in Southwest Pass. This account was augmented by discussions with personnel of the US Army Engineer District, New Orleans, to arrive at a general description of how the agitation dredging is performed. Nominal channel dimensions in the

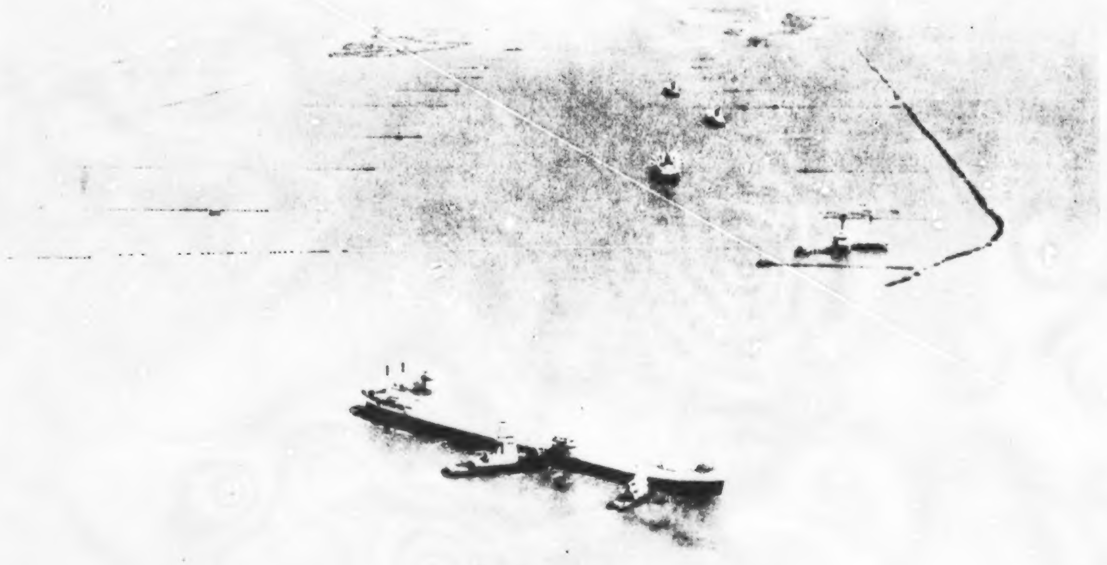

Figure A85. Southwest Pass, 1973

region of agitation dredging are 40 by 600 ft. The channel turns approximately 43 deg at the jetty tips, making maneuvering in this area difficult. A typical dredging sequence is to work the west side of the channel when entering the pass and the east side going out. Daily surveys determine where to dredge, and the operation is carried out around the clock during dredging season. The dredge master determines when currents in a given area are strong enough for agitation dredging and when to cease agitation and haul dredged material to a disposal site. When agitation is performed, material is carried out of the jetty tips by river surface currents, which do not follow the sharp bend in the navigation channel. Prevailing littoral currents flowing east to west help to carry agitated material away from the channel before it redeposits. Scheffauer presents a graph designed to aid in such an operation by identifying the largest size sediment particle that could reach a redeposition area in suspension, given the distance from the dredge to the area, current velocity, and water depth (Figure A86). A similar graph appears in the Corps of Engineers hopper dredge manual (Office, Chief of Engineers 1953), which states that it was developed by New Orleans District personnel using Stokes'

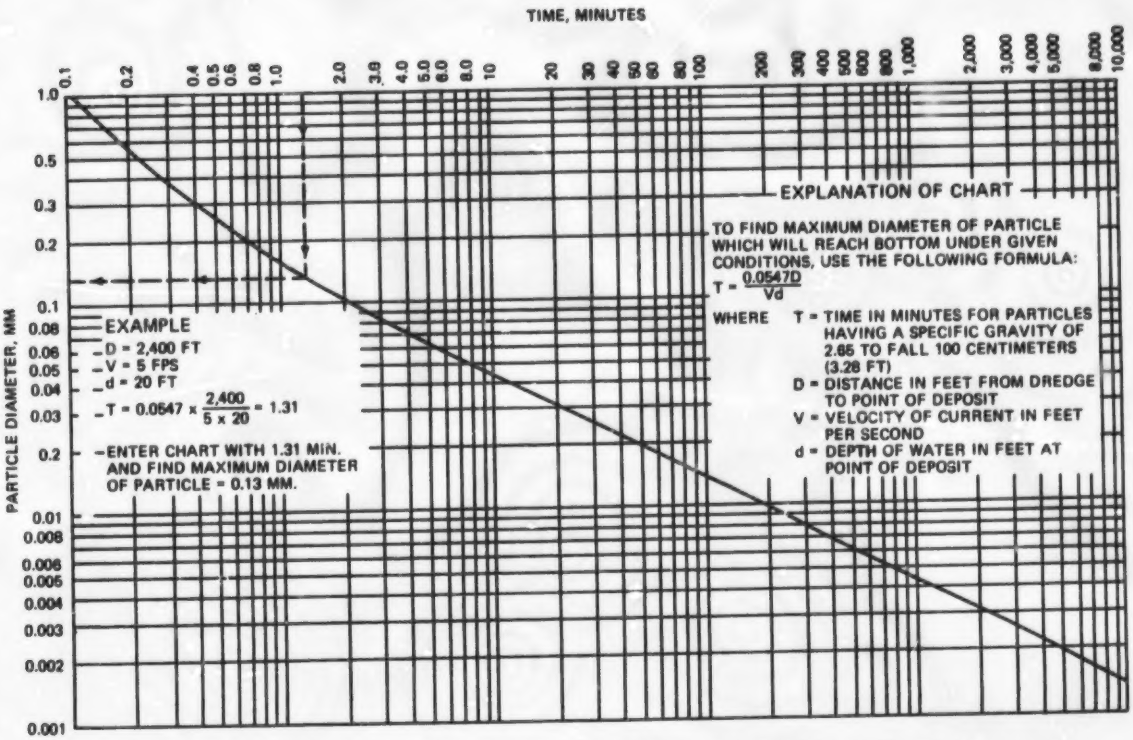

Figure A86. Chart for determining largest grain sizes to reach deposit area

relationship between particle size and settling velocity. Scheffauer uses this graph and information on sediment grain-size distributions in Southwest Pass to show that most material agitated therein will travel beyond the jetty tips with a river current of 5 fps or greater. Most of this sediment is finer than 0.15 mm, with typical median grain sizes of 0.1 mm in the channel between the jetties and 0.085 mm at the mouth.

135. During a typical operating day in Southwest Pass, the *Langfitt* will make one or two trips to dump material that settled in the hopper below the emergency gates. If such material remains in the hopper too long, it compacts and is difficult to remove. It is estimated that agitation dredging adds 4.5 pumping hours to a dredging day in Southwest Pass by minimizing total haul time per day. For example, in one period when 1,079 hr of dredging time were available, 82 percent of them were spent agitation pumping. Some redeposition of material in the channel can be tolerated to achieve such high pumping time percentages.

136. Agitation dredging in the Calcasieu River approach channel utilizes

westerly littoral currents to carry material away before it redeposits. The *Langfitt* and the Corps hopper dredge *McFarland* have been the primary equipment used. The channel is 42 by 800 ft, larger than Southwest Pass, and also considerably longer in terms of agitation dredging areas. Thibodeaux and Grimwood (1978) monitored agitation dredging in miles 4 to 7 of the approach channel to determine its effects on water quality. Water temperature, pH, conductivity, and dissolved oxygen were measured every minute for 40 min, starting 10 min before dredging commenced. Water samples were collected at the surface, 15-ft depth, and 30-ft depth prior to dredging, immediately after dredging, 1 hr later, and 2 hr later. Samples were also collected in the dredge hopper. These samples were analyzed to determine the levels of 74 different parameters relating to water quality. Temperature, pH, conductivity, and dissolved oxygen in the water column were virtually unchanged by the agitation dredging. The pH of water in the dredge hopper was slightly less than that in the surrounding area. Of the 74 parameters measured in water samples, eight were found that exceeded background levels significantly in the agitation dredging effluent: arsenic, chromium, copper, lead, nickel, zinc, total organic carbon, and suspended solids. All parameters returned to background levels 1 to 2 hr after dredging ceased. The greatest increase in most parameters occurred in the 15-ft-depth samples. Levels of all parameters were considerably less throughout the water column than in the dredge hopper.

137. Figures A87 and A88 summarize hopper dredging in the Mississippi River Passes and the Calcasieu River for the years 1965-1974. Figure A87 shows the total volumes dredged each year at each location by agitation and by hauling. The amounts vary considerably from year to year at each site and even between sites, with more total agitation in the Calcasieu River in some years than in the Passes. Relative amounts hauled at each site vary from year to year also. Tremendous increases in agitation dredging were needed in 1973 at both sites and in 1974 in the Passes due to unusually high river discharge. Virtually all hopper dredging in the Passes in 1973 was performed by agitation using the Corps of Engineers' hopper dredges *Langfitt*, *Gerig*, and *McFarland*. In this 1 year, the *Langfitt* agitated over 18 million cu yd in both the Passes and the Calcasieu River, for a total of almost 37 million cu yd agitated by one hopper dredge in 1 year. For 1974, the *Davison* and the *Essayons* were added to the Passes fleet, and the five Corps' hopper dredges agitated a total of 34.2 million cu yd and hauled 14 million cu yd in the Passes area.

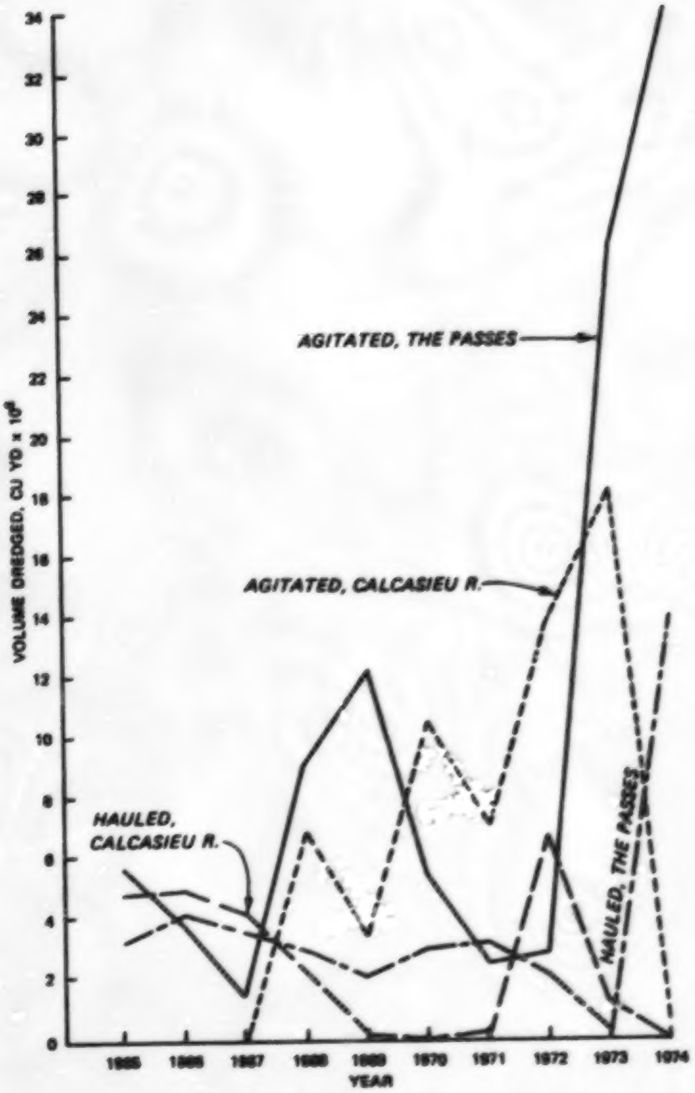

Figure A87. Hopper dredging, the Passes and Calcasieu River

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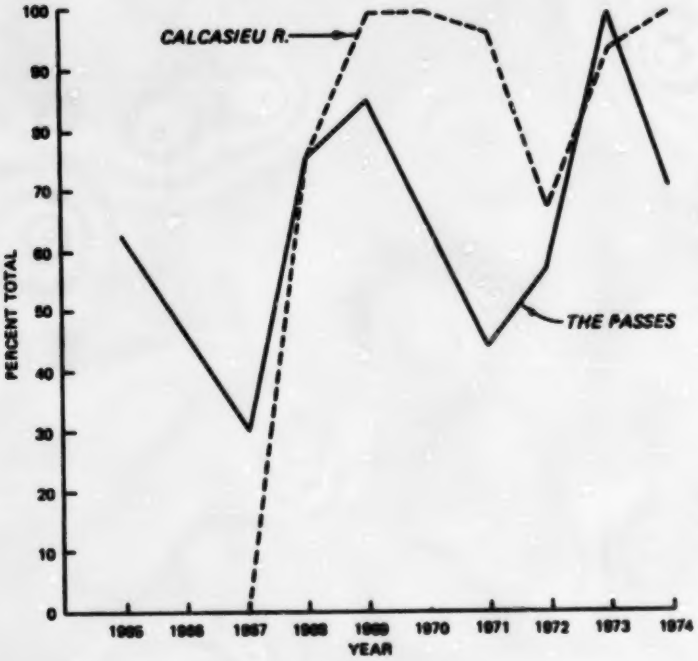

Figure A88. Agitation dredging as percent total hopper dredging, the Passes and Calcasieu River

138. Figure A88 plots agitation dredging in the Mississippi River Passes and the Calcasieu River as a percentage of total hopper dredging in these areas. Since 1968, it appears that agitation dredging has been relatively more important at the Calcasieu River than the Passes, although in most years over 50 percent of the Passes hopper dredging was accomplished by agitation. For 5 years (1969-1971 and 1973-1974) over 90 percent of the Calcasieu River hopper dredging was by agitation.

139. In discussing volumes of agitation dredging, it is necessary to know the method by which these volumes were measured. In the cases of the Passes and the Calcasieu River, the method is an indirect one outlined in the Corps of Engineers' hopper dredge manual (Office, Chief of Engineers 1953). A volumetric pumping rate under certain dredging conditions is calculated by measuring the hopper load obtained in a given time period without overflow. Then, this pumping rate is multiplied by the time spent agitating after overflow begins to arrive at agitation dredging volumes. Obviously, this method is approximate and does not account for material redepositing in the channel.

140. Table A6 summarizes performance and cost data for the *Langfitt* for agitation dredging channel maintenance in 1965-1972 (Office, Chief of Engineers 1965-1972).

## Table A6

## Performance and Cost Data, Hopper Dredge *Langfitt*, Maintenance Agitation Dredging

| Year | Average Sediment, $d_{50}$, mm | Average Cubic Yard per Effective Minute |  | Average Cost per Cubic Yard, Dollars |  |
|---|---|---|---|---|---|
|  |  | Hauled | Agitated | Hauled | Agitated |
| 1965 | 0.030 | 21.3 | 99.0 | 0.174 | 0.054 |
| 1966 | 0.053 | 31.9 | 98.9 | 0.1594 | 0.0514 |
| 1967 | 0.041 | 35.5 | 102.5 | 0.1569 | 0.0543 |
| 1968 | 0.069 | 28.2 | 99.9 | 0.225 | 0.064 |
| 1969 | 0.079 | 26.5 | 104.8 | 0.256 | 0.065 |
| 1970 | 0.058 | 27.6 | 108.1 | 0.269 | 0.069 |
| 1971 | 0.078 | 29.7 | 104.1 | 0.254 | 0.072 |
| 1972 | 0.042 | 42.9 | 116.6 | 0.188 | 0.069 |

141. The ratio of production per effective minute of dredging for agitation versus hauling ranged from 2.7 to 4.6 for the years covered in Table A6. Similarly, hauling costs per cubic yard were 2.7 to 3.9 times those for agitation. As a comparison, Table A7 lists similar data for selected Corps dredges in certain years at different locations around the United States, taken from the same sources as the data in Table A6.

## Table A7

## Performance and Cost Data, Selected Corps Hopper Dredges, Maintenance Agitation Dredging

| Dredge/ Year | Average Sediment, $d_{50}$, mm | Average Cubic Yard per Effective Minute |  | Average Cost per Cubic Yard, Dollars |  |
|---|---|---|---|---|---|
|  |  | Hauled | Agitated | Hauled | Agitated |
| *Gerig*/ 1972 | 0.25 | 17.7 | 83.0 | 0.566 | 0.121 |
| *Gerig*/ 1971 | 0.264 | 19.8 | 76.6 | 0.489 | 0.126 |
| *Gerig*/ 1970 | 0.315 | 20.4 | 55.3 | 0.373 | 0.137 |
| *Lyman*/ 1969 | 0.087 | 8.2 | 29.5 | 0.383 | 0.107 |
| *Hyde*/ 1969 | 0.313 | 6.3 | 14.9 | 0.790 | 0.332 |

142. Table A7 shows a range of production and cost ratios similar to the range experienced by the *Langfitt*, even though the material handled was generally much coarser. The costs of agitation dredging in a given year, however, are 1.6 to 5.1 times those for the *Langfitt*. This suggests that while hopper overflow agitation dredging often proves to be faster and cheaper than conventional hauling and dumping, its absolute cost may vary considerably from site to site. The *Langfitt* worked in the same locations and agitated many times the volume of material moved by the dredges in Table A7, giving her the advantages of experience and volume in addition to working in much finer material.

143. In mid-1982 the *Langfitt* was replaced by a new Corps of Engineers' hopper dredge, the *Wheeler*. Although it is only 57 ft longer than the *Langfitt*, it has 2.8 times the hopper capacity (8,400 cu yd versus 3,000 cu yd). It carries three drag arms, two on either side connected to one dredge pump and one in a center well connected to another pump. When agitating, all three drag arms are used. Hopper overflow levels are continuously variable, allowing optimum discharge heights for agitation disposal. The dredge is equipped with production meters, which will allow more accurate measurement of dredged volumes.

## Mare Island Strait

144. Scheffauer (1954) describes the formulation in the early 1920's of a hopper dredging program for Mare Island Strait which included hopper overflow agitation dredging. The general physical and sedimentary characteristics of Mare Island Strait were described earlier in this appendix in the Mare Island Naval Shipyard section.

145. In the period 1917-1927, the Mare Island Strait navigation channel was maintained at dimensions of 35 by 500 ft (Figure A89) and included a 1,000-ft-wide turning basin in the project upper half. The Corps of Engineers' hopper dredge *San Pablo* was constructed in 1916 to work in Mare Island Strait and the Pinole Shoal Channel. The *San Pablo* was 163 ft long with a hopper capacity of 525 cu yd. Two side-mounted drags connected to two 180-hp dredge pumps allowed a maximum dredging depth of 42 ft. The dredge was propelled by a single screw driven by a 700-hp steam engine, giving a speed of 6.9 knots light and 6.1 knots loaded. The *San Pablo* was unusual for its time in that it

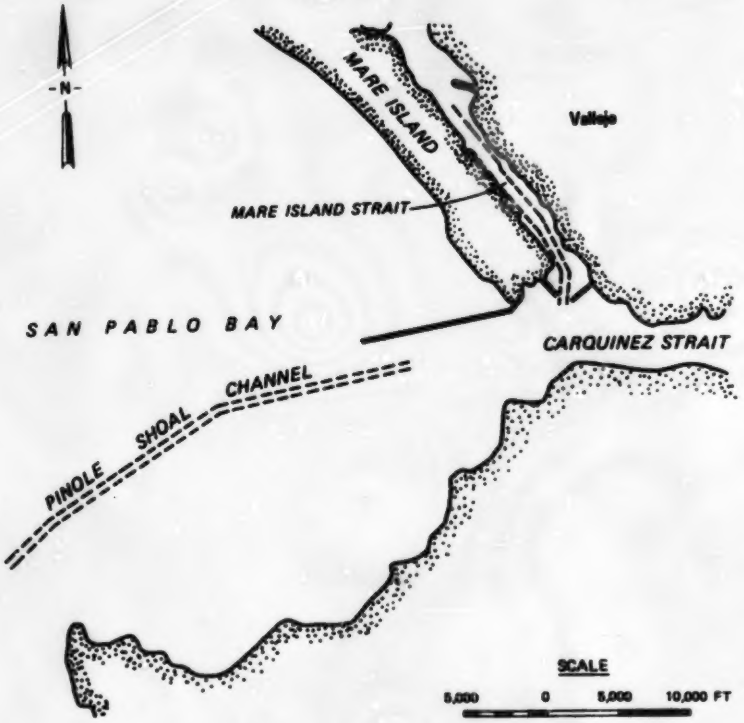

Figure A89. Mare Island Strait and Pinole Shoal Channel, 1920's

was equipped to pump out its hoppers through a pipeline and carried sliding-trunnion drag arms; both features are common on modern hopper dredges.

146. A series of tests were begun in 1922 to determine the best way of utilizing the *San Pablo* in Mare Island Strait. In earlier operations, it was found that hopper overflow began after only 5 min of pumping time, due to the fine, unconsolidated silty material being dredged. Prior to 1922, the *San Pablo* would pump an average of 51 min per load and spend 40 min hauling the load to a disposal site. In March-April 1922, the pumping time per load was reduced to 30 min. Obviously, a significant amount of overflow occurred with this operating schedule.

147. In the period May-June 1922, the operating schedule was modified to correspond with tidal flows in the strait. On flood tide, the dredge worked in the upper half of the strait pumping 30 min per load and hauling loads to dump disposal areas. During ebb tide, the dredge moved to the lower half of Mare Island Strait and repeated the same procedure, except that it pumped continuously (no dumping) during the stronger portions of spring ebb tides. In

July-August 1922, the pumping time per load was reduced to 15 min and material from the upper half of Mare Island Strait was pumped ashore instead of being dumped in a disposal area. During both time periods, channel shoaling increased markedly even though the *San Pablo* was handling more material than it had ever been able to. In May-June, the strait showed a net shoaling of 97,000 cu yd/month. In July-August, this increased to 221,000 cu yd/month. Although some of this could be attributed to increased natural sediment loads in the strait, the conclusion was that the dredge operating program on ebb tide should be modified to take advantage of ebb current flushing. Accordingly, a modified operating program was begun in July 1923 in which the *San Pablo* pumped 15 min per load and discharged ashore during flood tide and slack water, but pumped continuously with no dumping or pumpout during ebb tide. The dredge operated in the upper half of the strait during stronger ebb currents and the lower half during weaker ebb currents. The success of this new program was shown in the dredging periods July 1923-March 1924 and December 1924-March 1925, when the strait showed an average net improvement of 238,000 cu yd/month. By comparison, the program used before 1922 caused a net improvement per month approximately one-half that figure.

148. In contrast to Mare Island Strait, it was found that the *San Pablo* produced better results in Pinole Shoal Channel when operated with minimal hopper overflow, regardless of tidal currents. Material dredged from Pinole Shoal Channel contained a significant amount of fine sand, which redeposited in the channel instead of being carried away by tidal currents. Therefore, instead of pumping 60 min per hopper load, the *San Pablo* pumped 10 min per load and dumped material in an area parallel to the channel. The result was removal of almost twice as much material and a net channel improvement.

## APPENDIX B: DUSTPAN AND SHALLOW-DRAFT SIDECASTER DREDGES

1. This appendix briefly describes two types of dredges--the dustpan and the shallow-draft sidecaster--that are normally used in ways approaching pure agitation dredging. Both dredges transport bottom material a short distance horizontally through a pipeline before returning it to the water column; therefore they could be viewed as pipeline dredges with unusually short discharges. However, the material they release is often carried away from the dredging site by natural currents, which is a characteristic feature of agitation dredging. Since they are both important and unique tools in the dredging effort of the US Army Corps of Engineers, they need to be mentioned in any study of agitation dredging.

### Dustpan Dredges

2. The dustpan dredge was developed by the US Army Corps of Engineers in the late 1800's and early 1900's for use on the Mississippi River. It has evolved into one of the major tools for navigation maintenance on the lower Mississippi and its tributaries. It also is used in countries such as Argentina (World Dredging & Marine Construction 1972*) and Italy (Modesti 1974).

3. A dustpan dredge basically is a hydraulic pipeline dredge with some unique features derived from its highly specialized use. Figure B1 is a schematic of a dustpan dredge showing these features: (a) "dustpan" suction head, (b) self propulsion, and (c) short, steerable discharge pipeline. The suction head, which gives the dredge its name, has an opening 1 to 2 ft high and 30 to 40 ft wide. Water jets are located along the face of the suction head to aid in fluidizing the bottom material, which is usually sand or sandy silt. The suction head connects to the suction pipe through a "Y" fitting. Self-propulsion is provided so that the dredge can move between dredging sites and maneuver at each site without auxiliary equipment. The short (up to 3,000 ft) discharge pipeline is equipped with a controllable deflector plate at the end. By changing the angle at which the slurry discharge strikes the plate, the pipeline can be swung to various orientations relative to the dredge. The pipeline is supported on swiveling pontoons that align themselves with the direction of riverflow.

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* References cited in this appendix are included in the References at the end of the main text.

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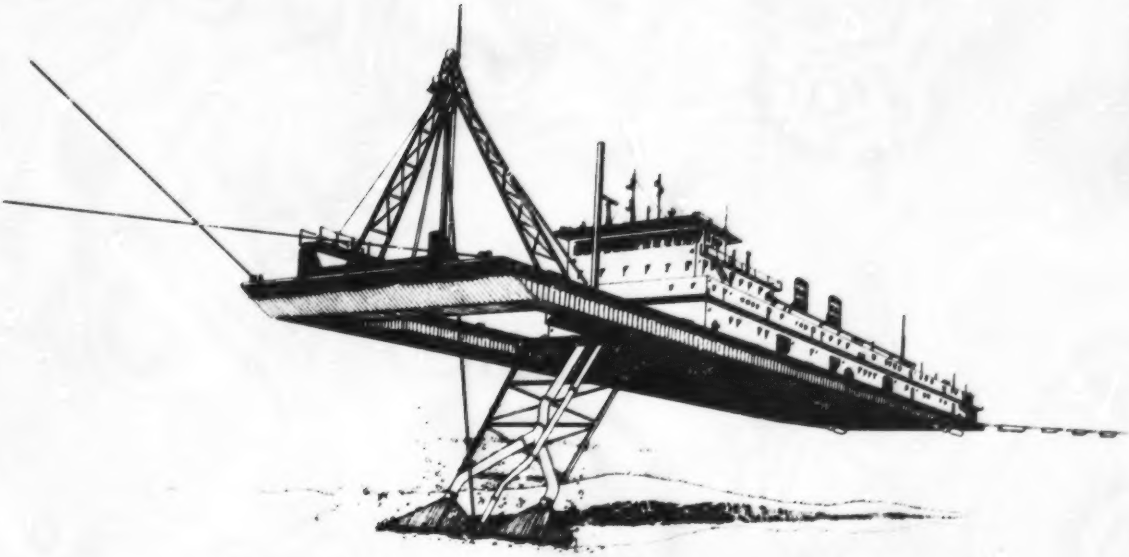

Figure B1. Dustpan dredge

4. The dustpan dredge operates facing in an upstream direction. Digging begins at the downstream end of a shoal and proceeds upstream in a series of cuts parallel to the channel length. River bottom currents aid in feeding material into the dustpan suction head and may transport sediment agitated by the head but not drawn into the suction. Unlike a conventional hydraulic dredging operation, discharge from a dustpan dredge is placed back into the river regime a relatively short distance away from the shoal. Depending upon conditions at the discharge point, the sediment may be carried a considerable distance farther by river currents. Since riverine shoaling often occurs at predictable, discrete locations such as bar crossings, this method of dredging and discharge can be very effective in maintaining navigation. Dredging rates can average 3,800 cu yd/hr for a large dustpan dredge and reach maximums of 4,800 cu yd/hr or more (Schmidt 1972).

## Shallow-Draft Sidecaster Dredges

5. The shallow-draft sidecaster dredge, like the dustpan, was a development of the Corps of Engineers. It was conceived as a means of maintaining the numerous small, shallow tidal inlets along the Atlantic coastline of North Carolina. It is also used to excavate "pilot" channels through shallow areas for hopper dredges. The original Corps development took place in the 1960's,

and an Australian sidecaster was built in the 1970's based on Corps experience (Perry 1978).

6. Figure B2 shows the design of a simple shallow-draft sidecaster dredge. Although a shallow-draft sidecaster has no provisions for transporting sand in hoppers, several large hopper dredges have been equipped with sidecasting booms as an alternate means of dredging. Sidecasters usually deposit material 80 and 90 ft away from the dredge hull. The longest sidecasting boom known is 430 ft on a hopper dredge in Venezuela (Marine Engineering 1960). Loaded draft of a shallow-draft sidecaster is usually in the range of 4 to 8 ft (Murden 1974) enabling it to work across ocean bars and inlet deltas. Suction equipment is similar to that found on a conventional trailing suction hopper dredge.

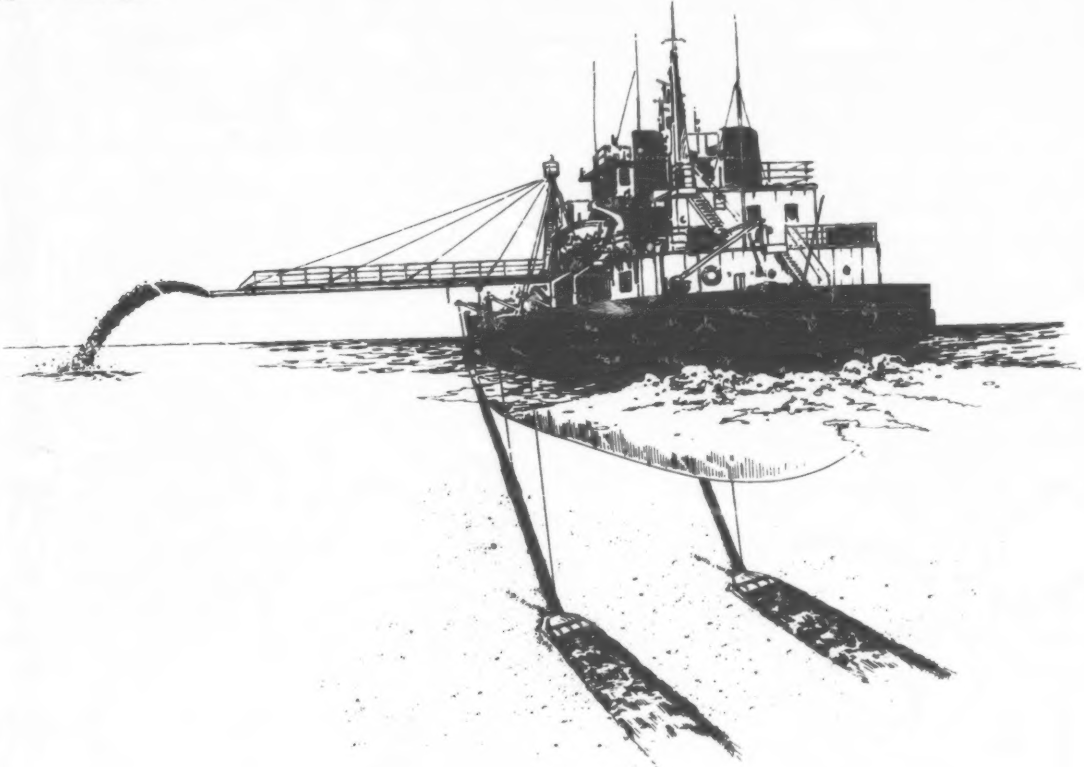

Figure B2. Shallow-draft sidecaster dredge

7. Long (1967) describes operating procedures and costs for a small shallow-draft sidecaster working on the North Carolina coast. Several modes of operation are mentioned, with the most efficient being a continuous loop cycle across the ocean bar, discharging toward the outside of the loop.

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Although this means that some of the dredged material is placed on the updrift side of the channel, it allows dredging to proceed as a relatively continuous operation and avoids the sometimes cumbersome operation of rotating the boom to discharge on the opposite side of the dredge. Material dredged by a sidecaster may also be transported by tidal and littoral currents after it leaves the discharge boom. Costs reported by Long for one shallow-draft sidecaster ranged from $0.27 to $0.75 per cubic yard in the period 1964-1966 and were considered comparable to pipeline dredge costs at similar locations.